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An Evaluation Study of the Design and Construction of an Existing Homogeneous Earth Embankment

A Thesis Submitted in Partial Fulfillment of the Requirements

For the Degree of Master of Science in Structural Engineering

By:

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DEDICATION

- ***To my family***

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ABSTRACT

The earth embankments are simple structures, but as any structure their design and construction of earth embankments require clear technical specification and qualified supervision and contractor staff to build a safe and stable structure.

In general the study presented in this thesis may be considered an effort to contribute the field of design, construction and evaluation of existing embankment.

The main objective of this study was represented in define the deficiencies in the existing embankment with respect to design and construction to evaluate its expected performance in the future.

The study reviewed the relevant data required for evaluation those available from the technical reports prepared by embankment designer, and additional data and information from a comprehensive inspection program planned and executed in this study to check and compare the data to be used in the evaluation process.

Also, the study presented the current condition of the embankment according to final results obtained from analysis of collected data.

The main conclusions drawn from this study that the evaluated earth embankment is neither suitable nor capable to protect the costly developments and structures planned to be constructed in the urban area on landside downstream of embankment. Extensive and costly remedial works would be done to reduce the risk of embankment failure and planning should be started to design and construct new structure, correctly and appropriately to protect the urban area.

الخلاصـــة

السدود الترابية منشات بسيطة التكوين لكن كأي منشات أخري يتطلب تصميمها وبناءها مواصفات فنية واضحة و مشرفين وبناءة مؤهلين لبناء منشاة ثابتة و أمنة.

عموما هذه الدراسة التي قدمت في هذه الأطروحة قد تعتبر مساهمة لإثراء مجال تصميم و تشييد وتقييم السدود الترابية.

الهدف الاساسى لهذه الدراسة تمثل في تعريف النقائص في سد ترابي منشأ فيما يتعلق بالتصميم و التشييد لتقييم أدائه المستقبلي.

استعرضت الدراسة المعلومات ذات الصلة و المطلوبة في عملية التقييم و المتحصل عليها من التقارير الفنية التي تم إعدادها بواسطة مصمم السد و المعلومات الإضافية التي تم الحصول عليها بتنفيذ برنامج بحث وتحقق شامل صمم ونفذ في هذه الدراسة للتحقق من و مقارنة المعلومات المستخدمة في عملية التقييم.

أيضا عرضت الدراسة الوضع الراهن للسد الترابي طبقا للنتائج النهائية المتحصل عليها من تحليل البيانات المجمعة.

الخلاصة الرئيسية التي توصلت لهل الدراسة إن هذا السد غير قادر على حماية المنشآت و التطورات القيمة المخطط لانشاءها في المنطقة الحضرية أسفل الحاجز الترابي, كما يجب القيام بمعالجات شاملة ومكلفة لتقليل خطر انهيار الحاجز الترابي كما لابد من بدء التخطيط لتصميم و بناء منشأة جديدة بصورة سليمة و ملائمة لحماية المناطق الحضرية

Table of Contents

List of Figures

Chapter 2. Stability, Safety and Evaluation of Embankment Dams; A Review

Chapter 3. Details of Study Program

Chapter 4. Analysis of Study Results

List of Tables

Chapter 2. Stability, Safety and Evaluation of Embankment Dams; A Review

Chapter 3 Details of Study Program

Chapter 4. Analysis of Study Results

CHAPTER (1)
INTRODUCTION

1.1 Introduction

Embankment dams are a momentous structures build to serve much human activity from economical and safety concepts. The construction of the embankment requires many resource and experience and takes much time; therefore it must be planned, designed and constructed under stringent standards to reach to good structure and high performance within safe operation to keep the human efforts.

The development in the human activity achieved firstly through science and deep studies on all field which supports and leads to progress. The search in the embankment design and construction represent the base of development in water and infrastructure projects. From this point comes the necessity of this study, also the deal with engineering of embankment dams need to know what to expect in any geological environment, and to have an understanding of the limitation of investigation and design method, and construction procedure. This study is considered to be representing the elementary step to enrich the field of design, construction and evaluation existing embankments in general and with particular reference to an embankment constructed for flood control in Khartoum state, Sudan.

1.2 Objectives of the Study

The general objectives and methodology of this study were as follows

i. Review the general literature in design and construction of the earth embankment and methods of evaluation.
ii. Evaluate the safety and stability of an existing embankment that constructed few years a go, and expect the performance of it in the future.

iii. Give comments and recommendations for existing embankment and same structure may be constructed or planned to build in the future.

1.3 Scope of the Study

The study follows the strategy and methodology summarized below:

i. Review of the useful studies and investigation in the embankment design, construction and evaluation, and adopt the comprehensive standard to establishing a phased design and evaluation program.
ii. Collect all available data related to the existing embankment considered in the present study.
iii. Execute the study program planned to obtain the missing and actual information and data for embankment under consideration.
iv. Analyze and discus the data collected from various sources and experimental program carried out.
v. Define the deficiencies in the existing embankment studied with respect to the design and construction to evaluate its expected performance in the future.

1.4 Layout of Thesis

Chapter 1 is an introduction to the study in which the objective and scope of this research work outlined.

Chapter 2 reviews the literature of stability, safety and evaluation of the embankment.

Chapter 3 is review of study program including the review of available data for existing embankment, and details of field data required for study.

Chapter 4 is analysis and discussion of the study results.

Chapter 5 is presentation of finding and conclusions of existing embankment, and recommendations for future studies.

Also this thesis attached by all results of tests was performed in the existing embankment.

CHAPTER 2

STABILITY, SAFETY AND EVALUATION OF EMBANKMENT DAMS; A REVIEW

2.1 Definition, Types and Selection of Embankment Dams

2.1.1 Introduction

An embankment dam may be defined as, “any dam constructed of excavated materials placed without addition of binding materials other than those inherent material. The materials are usually obtained at or near the dam site’’.

The purpose of embankment dam is to impound water for much utilization such as irrigation, protection and dams constructed to groundwater recharge.

A typical section of an embankment dam is show in Figure 2.1

2.1.2 Types of Embankment Dam

The two principal types of Embankment dams are earth and rock-fill dams, classified according to the predominant fill materials used for construction and described as follows.

2.1.2.1 Earth Dams

An earth dam is composed of suitable soil obtained from borrow areas or required excavation and compacted in layers by mechanical means. Typical sections of earth dams showing zoning for different types of quantities of fill materials of various methods for controlling seepage presented in Figure 2.2.

2.1.2.2 Rock-fill Dams

A rock-fill dam is one composed largely of rock with an impervious core. The core is separated from the rock by series of transition zones built of properly graded material. Two generalized sections of rock-fill dams are shows in Figure 2.3.

2.1.3 Selection of Dams Type

Site conditions that may lead to selection of an earth or a rock-fill dam rather than concrete dam (or rolled-compacted concrete dam) include a wide stream valley, lack of firm rock abutments, considerable depths of soil overlying bedrock, poor quality from structural point of view, availability of sufficient quantities suitable soils or rock-fill and existence of good site for a spillway of sufficient capacity.

The factors which may effect the selection of embankment include:

a) Topography: to a large measure, dictates the first choices of type of dam. A narrow V-shaped valley with sound rock in abutments would favor an arch dam. A relatively narrow valley with high rocky walls would suggest a rock-fill or concrete dam (or rolled-compacted concrete). Conversely, a wide valley with deep overburden would suggest an earth dam. Irregular valleys might suggest a composite structure, partly earth and partly concrete composite section also might also be used to provide a concrete spillway.

b) Geology and foundation condition: the geology and foundation conditions at dam site may dictate the type of dam suitable for that site. Competent rock foundation with relatively high shear strength and resistance to erosion and percolation offer few restrictions as to the type of dam that can be built at the site. Gravel foundation, if well compacted, are suitable for earth or rock-fill dams. Special precautions must be taken to provide adequate seepage control and/or effective water cutoffs or seals. Also, the liquefaction potential of gravel foundation should be investigated. Silt or fine sand foundation can be used for low concrete and earth dams but are not suitable for rock-fill dams. Clay foundation may be used for earth dams but require flat embankment slopes because of relatively low foundation shear strength. Because of requirement for flatter slopes and tendency for large settlement, clay foundations are generally not suitable for concrete or rock-fill dams.

c) Materials available: The most economical type of dam will often be one for which materials can be found within a reasonable haul distance form site. Materials which may be available near or on the dam site including soils for embankment, rock for embankment and rip rap, and concrete aggregate (sand, gravel and crushed stone). If suitable soil for an earth-fill dam can be found in nearby borrow pits, an earth dam may prove to be more economical.

d) Spillway: The size, type and restrictions on location of the spillway are often controlling factors in the choice of the type of dam. When a large spillway is to be constructed, it may be desirable to combine the spillway and dam into one structure.

e) Environmental factors: Recently, environmental considerations have become very important in the design of dams and can have a major influence on the type of dam selected. The principle influence of environmental concerns on selection of specific type of dam is the need to consider protection of the environment, which can affect the type of dam, its dimensions, and location of spillway and appurtenant facilities.

f) Economic factors: The final selection of the type of dam should be made only after careful analysis and comparison of possible alternatives, and after thorough economic analysis that include costs of spillway, power and control structures, and foundation treatment.

2.1.4 Comparison Between a Levee and an Earth Dam

The term levee is used to embankment whose primary purpose is to furnish flood protection from seasonal high water and which is therefore subject to water loading for periods of only a few days or weeks. Embankments that are subject to water loading for prolonged periods (longer than normal flood protection requirements) or permanently should be defined as an earth dam.

Even though levees are similar to small earth dams they differ from earth dams in the following

a) A levee embankment may become saturated for only a short period of time beyond the limit of capillary saturation.
b) Levee alignment is dictated primarily by flood protection requirements, which often results in construction on poor foundation.
c) Borrow is generally obtained from shallow pits or from channels excavated adjacent to the levee, which produce fill material that is often heterogeneous and far from ideal.

Selection of the levee section is often based on the properties of poorest material that must be used.

Levees are broadly classified according to the area they protect as either urban or agricultural levees because of different requirement for each. Urban levee that provide protection from flooding in communities, including their industrial, commercial and residential facilities. Agricultural levee that provide protection from flooding in lands used for agricultural purpose, also we can classify levee according to use, Table 2.1 below illustrates that:

Table 2.1 Classification of levee according to use

Type	***Definition***
Mainline and tributary levee	Levee that lie along a mainstream and its tributaries respectively
Ring levee	Levee that completely encircle or (ring) an area subject to inundation from all directions
Setback levee	Levee that are built landward of existing levee because the existing levee have suffered distress or are in some way begin endanger, as by river migration.
Sub levee	Levee built for the purpose of under seepage control, sub levee encircle area behind the main levee which are subject, during high-water stage, to high uplift pressure and possibly the development of sand boils. Sub levees are rarely employed as the use of relief wells or seepage berms make them unnecessary except in emergencies.
Spur levee	Levees that project from the main levee and serve to protect the main levee from erosive action of stream currents spur levees are not true levees but training dikes.

2.2 Geotechnical and Hydraulic Data and Information Required for Embankment Design

2.2.1 Introduction

The selection of dam location, dam structure, dam section, and fill materials depends completely on available data including the hydraulic data, geological data, topographical data, and materials properties. These data obtained through field investigation and laboratory tests, represent the main role in dams design process, and the sufficient and accurate data definitely leads to successful design process. Brief descriptions of ways of data collection are given in the following sections.

2.2.2 Field Investigation

2.2.2.1 General

Field investigation at the sites of structures and at possible borrow area must be adequate to determine suitability of the foundation, required foundation treatment, availability and Characteristic of embankment materials. Field investigation stages carryout by sequences of geological study, subsurface exploration, and field testing to provide enough data for following:

a) Sources of materials, for following purposes:

- Earth fill, impervious core.
- Filters.
- Rock fill
- Rip-rap.
- Concrete aggregate,
- Pavement.

For each material, the location of alternative sources, quantities, method for borrowing and processing, overburden and waste materials and quantities. Possible use of materials from required excavation e.g. spillway or outlet works.

b) Reservoir information

(c) Embankment information

- Location- to suit topographic and geological situations.
- Alternative sites for comparison of costs and geotechnical and other issues.
- Depths of suitable foundation for: earth fill, impervious core, filters; plinth or grout cap.
- Nature of materials to be excavated, excavation methods and possible uses of materials.
- Stability of excavations supports requirement.
- Permeability, compressibility and erodibility of foundation.
- Foundation treatment (s) required: slurry concrete, dental treatment, filter blanket; other.

d) Spillway, river diversion works and permanent outlet works

- Location and type.
- Stability of excavation, need for temporary/permanent support.

f) Seismicity of region (maximum credible earthquake).

Many field investigations are conducted in two stages: a preliminary stage and a final (design) stage. Normally, a field investigation in preliminary stage is not extensive since its purpose is simply to provide general information for project feasibility studies. It will usually consist of a general reconnaissance with only limited subsurface exploration of simple soil tests. In final (design) stage, more comprehensive exploration is usually necessary with more extensive geological reconnaissance, borings, test pits and possibly geophysical studies. Sometimes field test such as vane shear test, groundwater observations and field pumping test are necessary.

2.2.2.2 Geological Study

A geological study usually consists of an office review of all available geological information on the area of interest and an on-site (field survey). The office study begins with a search of available information, such as topographic, soil and geological maps and aerial photographs, pertinent information an existing construction in the area should be obtained, this includes design, construction and performance data or utilities, highways, railroads and hydraulics structure. Available boring logs should be used. Field survey is started after becoming familiar with the area through the office study. Visiting the proposed sites and borrow areas are always an excellent means of obtaining useful information.

2.2.2.3 Subsurface Exploration

The surface exploration for design stage generally is accomplished in two phases, which may separate in sequence, or concurrent: phase1, the main purpose of which is to better define the geology of the area, the soil types present and to develop general ideas of soil strength and permeability, phase 2, provides additional information an soil type present and consist of both disturbed and undisturbed sample borings and also may include geophysical method. Undisturbed sample sometimes obtained by hand carving block samples from test pits but more usually by rotary and bush-type drilling methods. Samples for determining consolidation and shear strength characteristics and values of density and permeability should be obtained using undisturbed boring.

The Subsurface Investigation techniques are

- Test pits and trenches.
- Drill holes.
- Geophysical exploration.

The spacing of boring and test pits in phase 1 is based on examination of geological condition determined in the preliminary stage or known from previous experience in the area, and by nature of the project. Initial spacing of the boring usually varies from 60-300 m, being closer spaced in expected problem areas and wider spaced in non-problem areas. The spacing should not be arbitrarily uniform but rather should be based on available geologic information boring are normally laid out along the dam centerline but can be staggered along the alignment in order to cover more area and to provide some data on nearby borrow materials.

In phase 2, the locations of additional general sample boring are selected based on phase 1, results.

Depth of boring along the alignment should be at least equal to the height of proposed dam at its highest point but not less than (3m). Boring depth should always be deep enough to provide data for stability analysis of the dam and foundation. Where pervious of soft material are encountered, borings should extend through the permeable material to impervious material or through the soft material to firm material. Borings at structure locations should extend well below foundation elevations and below the zone of significant influence created by the load.

In borrow areas, the depth of exploration should extend several meter below the allowable borrow depth or to groundwater table, if borrow is to be obtained from below the groundwater table, boring should be at least (3m) below bottom of the proposed excavation.

2.2.2.4 Field Testing

In many cases it is preferable to measure the properties of soil and rock in dam foundation using in site tests, rather than taking samples and testing in the laboratory. In some cases (e.g. estimation of the relative density of sand) in situ testing is the only method available.

The most commonly used is situ tests in soil and their applicability to dam engineering are

- Standard penetration test (SPT).
- Static cone penetration test (CPT).
- Vane shear test.
- Field pumping test.

2.2.3 Laboratory Tests

The purpose of laboratory tests is to investigate the physical and hydrological properties of natural materials such as soil and rock which are used in dams, determine index value for identification and correlation by means of classification tests, and define the engineering properties in parameters usable for dams design.

Laboratory test programs for embankment dams will vary from minimal to extensive, depending on the nature and importance of the project and on the foundation conditions, and whether existing experience and correlation are applicable. Testing programs generally consist of water content and identification tests on most sample and shear, consolidation and compaction tests only on representative samples of foundation and borrow materials. It is imperative to use all available data such as geological and geophysical studies, when selecting representative samples for testing. Soil tests that may be included in laboratory testing programs are listed in Table 2.2 a and b, together with pertinent remarks on purposes and scope of testing.

Table 2.2 a Laboratory testing for fine-grained cohesive soils

Test	Remarks
Visual classification and water content determinations	On all samples
Atterberg limits	On representative samples of foundation deposits for correlation with shear or consolidation parameters, and borrow soils for comparison with natural water contents, or correlations with optimum water content and maximum densities.
Permeability	Not required; soils can be assumed to be essentially impervious in seepage analyses
Consolidation	Generally performed on undisturbed foundation samples only where: **a.** Foundation clays are highly compressible. **b**. Settlement of structures within dam systems must be accurately estimated. Sometimes satisfactory correlations of Atterberg limits with coefficient of consolidation can be used. Compression index can usually be estimated from water content.
Compaction	**a.** Required only for compacted or semi-compacted embankment.

Table 2.2 a- continue

Test	Remarks
	b. Where embankment is to be fully compacted. Perform standard 25-below compaction tests. **c.** Where embankment is to be semi-compacted Perform 15-below compaction tests.
Shear strength	-Unconfined compression tests on saturated foundation clays without joints or slickenside. -Q triaxial tests appropriate for foundation clays, as undrained strength generally governs stability. -R triaxial and S direct shear: Generally required only when foundations are weak, or at locations where structures exist in dams. -Q, R and S tests on fill materials compacted at appropriate water contents to densities resulting from the expected field compaction effort.

Table 2.2 b Laboratory testing for pervious material

Test	Remarks
Visual classification	On all jar samples
In situ density determinations	Of Shelby- tube samples of foundation sands where liquefaction potential must be evaluated.
Relative density	Maximum and density tests should be performed in seismically active areas to determine in situ relative densities of foundation sands and to establish density control of sand fills.
Gradation	On representative foundation sands: –For correlating grain-size parameters with

Table 2.2 b- continue

	permeability or shear strength. -For size and distribution classifications pertinent to liquefaction potential.
Permeability	Not usually performed. Correlations of grain-size parameters with permeability or shear strength used. Where under seepage problems are serious, best guidance obtained by field pumping tests.
Consolidation	Not usually necessary as consolidation under load is insignificant and occurs rapidly.
Shear strength	For loading conditions other than dynamic, drained shear strength is appropriate. Conservative values of Ǿ can be assumed based on S test on similar soils. In seismically active areas cyclic triaxial tests may be performed.

2.3 General Design and Construction Consideration

2.3.1 Introduction

The design of an embankment dams is complex because of the unknown of the foundation and materials available for construction. Past experience confirms that embankment dams can easily be (tailor-made) to fit the geologic site conditions and operational requirements for a project.

Moreover experience have always played a significant role in the design of embankment dams, the detailed analyses should be performed using arrange of variables to allow an understanding of the sensitivity of the particular analysis to the material properties and the geometric configuration.

An understanding of the causes of failure is a critical element in the design and construction process for new dams and for the evaluation of existing dams. The

failure of previous dams may be consider effort in the area of dam safety and dams design, and create the inventory at design consideration and guides.
This chapter covers the design concept from and through the failure modes and their likelihood of occurrence, and the general considerations and limitation must be taken in dam design and construction process.

2.3.2 Failure Mode Analysis

The project requirements, geologic assessment and site characterization, unique project features, loading conditions, and design criteria for the dam and appurtenant structures are the basis for the detailed project design. As the design progresses, an assessment of the materials distribution is made and a preliminary embankment section is established. The next step is to conduct a preliminary failure mode analysis, this consists of identifying the most likely modes of failure for dam, foundation, abutments and appurtenant structures as designed. It is important to have a through understanding of the historic causes of failure and their respective probabilities of occurrence, the failure modes should then be listed in the order at their likelihood of occurrence. During the final design, the failure modes are reviewed and updated.
On the basis of investigation reports on most of the past failures, it has now been possible to categories the type of failures into three main classes

- Hydraulic failure
- Seepage failure.
- Structure failure.

2.3.2.1 Hydraulic Failure

Hydraulic failure including the following:

a) Overtopping: the earth dam may get overtopped if the design flood is underestimated, or if the spillway is of insufficient capacity. Also insufficient freeboard or settlement of foundation and embankment may also lead to overtopping.

b) Toe erosion: toe erosion my occur due to two reasons (1) erosion due to tail water and (2) erosion due to cross- currents that may come from spillway buckets or from exist areas of outlets. The toe erosion can be avoided by providing thick riprap on the downstream, up to height slightly above the tail water level.

c) Gullying: downstream slope may fail due to the formation of gullies by heavy downpour. To eliminate failure due to gullying proper turfing and good drainage system should be provides to the downstream side.

d) Wave erosion: The effect of wave is to notch out earth from upstream slope in absence of proper slope protection in the form of riprap.

2.3.2.2 Seepage Failure

Seepage failure may be due to (a) piping and (b) sloughing.

(a) Piping: The seepage of water through the body and foundation of the earth dam lead to piping or progressive erosion of concentrated leak, causing a large number of catastrophic failures.

The bad effects of seepage through the embankment represent in

i) Seepage water generates erosive forces which remove particles from the soil structure and case migration of the fines to voids between larger grains.

ii) The flow with its associated differential pore pressure can lift portion of the soil mass causing boiling.

iii) Internal erosion of the soil mass, progressive back words from the point of exit leads to formation of an open conduit (channels) through the soil.

iv) The internal pressure in the soil water can reduce that part of the soil strength that is developed by internal friction and thereby leads to weakening of the soil mass and even failure by shear.

Probably the most common cause of embankment failure had been poor construction control which can result in inadequately compacted or pervious layers in the embankment, inferior compaction and bond between the embankment and foundation or abutments. Embankment leaks through differential settlement cracks have also been a major source of trouble.

(b) Sloughing: Failure due to progressive sloughing or raveling is closely related to piping. Under the full reservoir condition the downstream remains saturated, and may erode producing a small slump or slide. This miniature slide leaves a relatively steep face, which becomes saturated by seepage from the reservoir and slump again, formation a slightly higher and more unstable face. This traveling process can continue till the remaining portion the dam is too thin to withstand the water pressure and complete failure occurs suddenly. **2.3.2.3 Structural Failures**

The structural failure may be due to the following reasons

1. Upstream and downstream slope failure due to construction pore pressure

When a dam is built of relatively impervious compressible soil, the drainage is extremely slow and excess pore pressure developed during and immediately after construction which affecting the soil strength.

2. Upstream slope slide during sudden drawdown

For the upstream slope the critical condition is when the reservoir suddenly emptied without allowing any applicable change in the water level within the saturated soil mass, this stage is known as sudden drawdown. With complete drawdown, the hydrostatic force acting along the upstream slope at the time of full reservoir is removed without the hydrostatic pressure on the slope to counteract it.

3. Downstream slope slide during full reservoir condition

Critical condition for downstream slope occurs when the reservoir is full and percolation is at its maximum rate. The direction of seepage tend to decrease stability, or in other words, the pore water pressure acting on soil mass below the saturation line reduce the effective stress which is responsible for mobilizing shearing resistance.

4. Foundation slides (spontaneous liquefaction)

When the earth dam has foundation of fine silt or silt soil, it can slide wholly sometimes, soft and weak clayey seam exist under the foundation and the dam can slide over it causing a failure. Excess water pressure on confined sand and silt seams in the foundation may also cause unbalanced conditions causing foundation failure.

5. Failure by spreading

Failure by spreading has been observed only in connection with fill located above stratified deposits that contain layers of soft clay.

6. Failure due to earthquake

7. Slope protection failure

Slopes are generally protected by riprap (either hand placed or dumped) over a layer of gravel or filter blanket. During a heavy storm the waves on the surface of reservoir hit repeatedly against the slope just above reservoir level. This action may have two effects: (1) the wave my pass through the voids of riprap and wash away the filter layer, settling the riprap layer and exposing the embankment to wave erosion. (2) if the average size of riprap not heavy it may be washed out of the layer by the hydraulic force generated by the waves.

2.3.3 General Design Considerations

2.3.3.1 Embankment Design Criteria

An earth dam must be safe and stable during phases of construction and operation of the reservoir. The practical criteria for the design of earth dam may be stated briefly as follows

a) The dam must safe against overtopping during flood by the provision of sufficient spillway and out let works capacity.
b) The dam must have sufficient free board so that it is not overtopped by wave action.
c) The seepage line should be well within the downstream face so that no sloughing of the slope takes place.
d) Seepage flow through the embankment, foundation and abutment must be controlled by suitable design provision so that no internal erosion takes place.
e) The portion of the dam downstream of the impervious core should be properly drained.

f) The upstream and downstream slopes should be so designed that are safe during all reservoir level.
g) The upstream and downstream slopes of the dam should be flat enough so that shear stress induced in the foundation is enough less than shear strength of the material in the foundation to ensure a suitable factor of safety.
h) The dam as a whole should be earthquake resistant.
i) The upstream and downstream slope must be protected against erosion.

The above criteria of design have been covered at length in the subsequent articles.

2.3.3.2 Embankment Materials

While most soil can be used for embankment construction as long as they are insoluble and substantially inorganic, typical rock flours and clays with liquid limits above 80 should generally be avoided. Fine grained soil with the range of water contents suitable for compaction and for operation of construction equipment; it can be used for the embankment construction. Some slow-drying impervious soil may be unusable as embankment fill because of excessive moisture, and the reduction of moisture content would be impracticable in some climatic areas because of anticipated rainfall during construction, in other cases, soil may require additional water to approach optimum water content for compaction.

Also soils having a wide range of grain sizes (well-graded) are preferable to soil having relatively uniform particle size. In general, soil having less susceptible to piping, erosion, liquefaction and less compressible is preferred.

Sound rock is ideal for compacted rock- fill, and some weathered or weak rocks may be suitable, including sandstone and cemented shales (but not clay shales).

Rocks that break down to fine sites during excavation, placement, or compaction are unsuitable as rock –fill and such materials should be treated as soil.

The embankment should be zoned to use as much material as possible from required excavation and from borrow areas with the shortest haul distance and the least waste. Embankment zoning should provide an adequate impervious zone, transition zones between the core and the shells, seepage control and stability.

Table 2.3 gives recommendation for suitability of soil used for earth dams as India standard 8826-1978.

2.3.3.3 Embankment Section

The preliminary design of an earth embankment is done on the basis of past experience and on the basis of the performance of the dam built in the past. The following items represent the preliminary section

(i) Top width.
(ii) Free board.
(iii) Slopes.
(iv) Central impervious core.
(v) Cut off trench.

(i) Top width

The crest width of an earth embankment depends as following considerations

- Nature of the embankment materials and minimum allowable percolation distance through the embankment at the normal reservoir level.
- Height of the structure.
- Width of highway on the top of the embankment.

- Practicability of construction.
- Protection against earthquake force.

The following empirical expressions can be used to estimate the width (b) of earth embankment.

b = $\frac{Z}{5} + 3$ applicable for very low embankment.

b = $0.55Z^{\frac{1}{2}} + 0.2Z$ applicable for embankment lower than 30m.

b = $1.65(Z + 1.5)^{\frac{2}{3}}$ given by U.S Bureau at Reclamation for embankment lower than 30m.

Where Z = the embankment height.

(ii) Freeboard

Freeboard is the vertical distance between the horizontal crest of embankment and the reservoir level. Normal free board is the difference in the level between the crest or top of embankment and normal reservoir level. Minimum freeboard is the difference in the elevation between the crest of the dam and the maximum reservoir water surface that would result should the inflow design flood occur and should the outlet works and spillway function as planned. Sufficient freeboard must provide so that there is no possibility whatsoever at the embankment being overtopped.

Table 2.4 shows the U.S Bureau of Reclamation suggested for freeboard. To estimation of freeboard Appendix A illustrates the general method which is based on wave height and wind velocity.

(iii) Slopes

The design slopes of the upstream and downstream embankments may vary widely, depending on the character and the materials available, foundation conditions and the height at the dam. The slopes also depend upon the type at the dam.

Table 2.5 gives preliminary side slopes of earth dam according to Terzaghi.

(iv)Central impervious core

The width of the core at the crest of the dam should be a minimum of 3m to permit economical placement and compation.The top level of the core should be at least 1m above the maximum water level to prevent seepage by capillary. Also the width of the core at the base cutoff should be equal to or greater than 25% of the maximum difference between the maximum reservoir and minimum tail water.

(v)Cutoff trench

Cut off is required to (a) reduce loss of stored water through foundation and abutment, and (b) prevent subsurface erosion by piping. Following are the Indian standard recommendations (IS: 8826-1978)

1. The positive cutoff should be taken at least one meter into continuous impervious sub stratum.
2. The side slopes depend upon sub-strata, side slopes of at least 1:1 or flatter may be provided in case of over burden, while ½ : 1 and ¼: 1 my provided in soft rock and hard rock respectively.
3. The bottom width of cutoff trench may be fixed taking the sufficient working space for compaction and piping safety, into consideration.
4. The backfill material for cutoff trench shall have same properties as those prescribed for impervious core.

2.3.3.4 Foundation Preparation

The degree of foundation preparation which is necessary for a dam embankment depends on

- The type of dam.
- The height at dam.

- Topography at the dam site.
- Erodibility, strength permeability, compressibility of the soil or rock in the dam foundation.

Generally foundation preparation depends on result the site investigation and materials properties of soil available in dam site, and it consists of

- Clearing.
- Grubbing to remove stumps and any vegetative roots.
- Stripping to remove sod, topsoil, boulders, organic materials, rubbish fill, highly compressible soils occurring in thin surface layer, and other undesirable materials.
- Flatting the abutment slopes and foundation to prevent and decrease the possibility of differential settlement which causes cracks.

After applying the previous works, the foundation surface will be in a loose condition and should be compacted.

Also weak and compressible liquefiable foundation must strengthen by using many improvement methods like grouting, compaction, wick drains and slow construction and or stage construction.

2.3.3.5 Stability of Slopes

i. Methods of analysis

The principal methods used to analysis the stability of embankment slopes against shear failure assume either (a)a sliding surface having the slope of a circular arc within the foundation and or the embankment ,(b)a composite failure surface composed of a long horizontal plane in a relatively weak foundation or thin foundation stratum connecting with diagonal plane surface up through the foundation and embankment to the ground surface.

Various methods of analysis can be chosen for use where determined appropriate by the designer. Computer programs are available for these analyses, with the various loading cases to which embankment and its foundation may be subjected. Appendix A illustrates the Swedish Circle Method which is used in slopes stability analysis and various loading case considered in embankment design.

ii. Conditions Requiring Analysis

The various loading conditions to which an embankment and its foundation may be subjected and which should be considered in analysis are designated as follows

(a) End of construction: this case represents undrained conditions for impervious embankment and foundation soil; excess pore water pressure is present because the soil has not had time to drain since being loaded. Result from laboratory Q (unconsolidated-untrained) test are applicable to fine- grained soils load under this condition while result of S (consolidated- drained) test can be used for pervious soils drain fast enough during loading so that no excess pore water pressure is present at the end at construction.

(b) Sudden drawdown: this case represents the condition where by a prolonged flood stage saturates at least the major part of the upstream embankment portion and then falls faster than the soil can drain. This causes the development of excess pore water pressure which may result in the upstream slope becoming unstable. Composite of results of consolidated-drained (S) tests (either direct shear or triaxial) and results of consolidated-untrained (R) tests can be used to analysis the stability of upstream slope.

(c) Steady seepage from full pool level (fully developed phreatic surface). This condition occurs when water remains at or near full pool level long enough so that the embankment becomes fully saturated and a condition of steady seepage occurs. This condition may be critical for downstream slope stability. Results of (S) tests can be used in this case.

iii. Minimum acceptable factor of safety

There are no (rules) for acceptable factor of safety in slope analysis, Table 2.6 illustrates the factors of safety recommend by US Corp of Engineers.

Table 2.6 US Corps of Engineers factor of safety for embankment dam

Case	Loading condition	Required factor of safety *
1	End of construction	≥ 1.3 for upstream and downstream slope
2	Steady seepage	≥ 1.5 for downstream slope
3	Rapid drawdown	≥ 1.2 for upstream slope

* Ratio at available strength to shear stress, required for stable equilibrium

2.3.3.6 Seepage Control

All earth and rock fill dams are subject to seepage through the embankment, foundation and abutments. Seepage control is necessary to prevent excessive uplift pressure, instability of downstream slope, piping through the embankment and/or foundation, and erosion of material by migration into open joints in the foundation and abutments. The purpose of the project, i.e. long –term storage, flood control, etc. may impose limitation on the allowable quantity of seepage.

The following devices are used for seepage control through the earth dam

i. Embankment Seepage Control

1. Toe filter.
2. Horizontal drainage filter.

3. Protective filter downstream of the toe.
4. Downstream coarse section (embankment zoning).
5. Chimney drains extending upward into embankment.

ii. Foundation Seepage Control

6. Impervious cutoff.
7. Upstream impervious blanket.
8. Downstream seepage berms.
9. Drainage trench.
10. Relief wells.

Figure 2.4 and 2.5 shows the previous seepage devices.

2.3.3.7 Filter Criteria

The dimensions and permeability of filter drains must be chosen in such a way that the drainage system can carry away the anticipated flow with an ample margin of safety. Considerable experimentation has been performed by the U.S Corps of Engineers and the U.S.B.R. generally, a multi-layer is provided, in which each subsequent layer becomes increasingly coarse than the previous one.

2.3.3.8 Slopes Protection

The upstream slope should be protected against the wave action and the downstream slope should be protected against rain. In same instances, provision must be made against burrowing animal. Following are the various materials used for slope protection

1. Rock riprap.
2. Concrete pavement.
3. Steel facing.

4. Precast concrete blocks.

The U.S Bureau of Reclamation has found a thickness of about 1m of dumped riprap, placed on 30 to 50 cm of gravel filter to be quite satisfactory for major dams. A layer of graded gravel should be provided under-neath the riprap when the compacted materials of underlying earth fill is of such gradation that there is danger that fines may be washed out through the voids in the riprap by wave action.

2.3.4 Embankment Construction

2.3.4.1 General

The design of an embankment is a continuous process, where the design continued until construction is complete. Ensure it must be that the new information acquired during construction to the assimilated into design, and ensure that the project is constructed according o the design.

Much additional information obtained during construction such as characteristic of foundation , and also operation in borrow area gives information about fill material , for that project must be continuously evaluated and re-engineered as required , during construction , to ensure that the final design is compatible with conditions encountered during construction.

Hereunder , the overall review of general construction concept for embankment and its component.

2.3.4.2 Foundation Preparation

The foundation preparation consist of many activity described in section 2.3.3 and by the ends of this activity foundation must be

-No organic material in foundation.

-dry enough.

-No any undesirable underground features such as old drain tile, sewer lines, animal burrows and pockets of undesirable materials,

- Satisfactory from strength, permeability and erodibility.
- Well compacted surface

2.3.4.3 Earth fill Placement

Embankment (and levee embankment) classified according to construction method used.

Category	Construction method	Use
I. Compacted	Specification: a.water content range with respect to standard effort optimum water content b.Using suitable compaction equipment with number of passes to attain a given percent compaction based on standard maximum density.	- Provides embankment section occupying min space provides strong embankment of low compressibility needed adjacent to concrete structure. - Requires strong foundation of low compressibility and availability of borrow materials with naturals with natural water content reasonably close to specified ranges.
II. Semi-compacted	- No water content control [compaction of fill material of their natural water content] - Wet borrow material would require drying before placement - Compacted evaluative to is-blow	Use where: - These are no severe space limitations and steep-sloped category I embankment are not required - Relatively weak foundation

	compaction test	- Under seepage condition are such as required wide base - Water content of borrow materials or amount of rainfall during construction is such as not to justify category I compaction
III. Un-compacted	Fill cast or dumped in place in thick layers with little or no spreading or compaction	Temporary emergency

For assessment, evaluation and quality control process the specification of fill material must be clear and reachable. The common specifications for earth fill for embankment are

- The sources of the earth fill.
- The maximum partial size of hard clay, gravel or rock fragment in the earth fill.
- The practical size distribution (minimum percentage passing 0.075 mm, minimum percentage passing 4.75 mm for ensure the earth fill in not gap-graded).
- An upper limit on fines (ensure the earth fill grading is compatible with design of filters) this may be done by specifying the grading envelope or as a limiting percentage passing the 0.075 mm sieve.
- Atterberg limits, this may reflect the presence of particularly high plasticity clays in borrow area which may be hard to compact.Atterberg

limits requirement should be seen only as away of allowing rejection of unsuitable material during construction.

- Layer thickness, water content and density ratio (for compaction process and controlling).
- Roller type and weight.

2.3.4.4 Filters Construction

Generally, the basic requirements of filters are

- Meet the design specification (material type, graduation, size, and compatible with other material used to construct embankment.
- Sufficiently fine grained to prevent erosion of the soil they are protecting.
- Sufficiently permeable to allow drainage of seepage water.

The common compaction specifications for filter are

- Horizontal filters can be placed in layer as 150 mm or as thick as 500 mm.
- Density index 70-80% and may reduced to 60%.
- Density index > 80% is likely to result in excessive breakdown of filters under the compactive effort.

2.3.4.5 Quality Control in Embankment Construction

Generally there are two principal types of technical specification

a. Method or procedure specification which describe how the construction is to be carried out, in order to achieve the desired end product.

   e.g: Specification of earth fill listed in section 2.3.4.3.

b. Performance or end product specifications which describe the end result to be achieved.

   e.g. specify the following in final product

- Partial size gradation
- Atterberg limits
- Water content
- Density ratio

Many dam specifications are a mixture of these two alternatives and absent of specification to inefficient construction procedures, and so lead to undesirable products.

The main three phases of quality control which must be considered in embankment construction are:

a. Inspection: The inspection is an important part of quality control plan. The field and laboratory testing program should be seen as first establishing the methods required to achieve the required quality.

   It is important that the inspectors are properly trained briefed on the implications of substandard work, and also the inspectors will often be needed in the borrow area, as well as on the embankment, so that unsuitable material can be rejected before it reaches the embankment.

b. Testing: testing carried out to ensure that the requirements of the specification are being meet. The selection of areas or materials for testing may be done in two ways:

i. Areas judged by the supervisor or inspector, this can reduce the quantity of testing.

ii. Selecting test areas at random, at the minimum recommended frequency. This method is better suited to establishing statistical limits to the testing, allowing recognition of the fact that there is a statistical sampling error, and that within a large mass of earth fill, a small proportion of the material failing to meet the basic specification criteria will not effect overall performance.

3. Reporting: It is important that complete records should be kept of all construction operation. These are invaluable in the event that repairs or modifications are required. The reporting should include:
   - Plans and specification, including amendments and work as constructed.
   - Find construction report written by the engineer.
   - Monthly progress reports.
   - Laboratory test reports, including clear definition of location and level of samples tested.
   - Daily reports.

2.3.4.6 Embankment Construction Deficiencies

Some of things that can happen during construction that can cause failure or distress of even low embankment on good foundation are given below

Deficiency	Possible
Organic material not stripped from foundation	Differential settlement, shear failure, internal erosion caused by through seepage
High organic or excessively wet or dry fill	Excessive settlement, inadequate strength
Placement of pervious layers extending completely through the embankment	Allows unimpeded through seepage which may lead to internal erosion and failure
Inadequate compaction	Excessive settlement, inadequate strength through seepage

Inadequate compaction of back fill around structures in embankment	Excessive settlement, inadequate strength, provide seepage path between structure and material which may lead to internal erosion and failure by piping

2.4 Evaluation of Existing Earth Embankments

2.4.1 Objectves of Embankment Evaluation

The evaluation of safety of both new and existing embankment dams presents special and unique problems. Existing dam may pore difficult to analyze especially in those instances where the embankment designed before the development of modern design and construction technology or where adequate records are not available. Even for a relatively new dam, where records are extensive.

Evaluation can be cumbersome for the following reasons

i. Various levels of completeness of records.
ii. Different site conditions.
iii. Varying degrees of quality in design and construction.
iv. Differing depth of evaluation required for each dam.

The main objectives of embankment evaluation are

- To provide confirmation of design assumptions
- To predictions of early warning of the development of unsafe trends in behavior.
- To make assumptions of remedial and stabilizing works or other measures.

2.4.2 Embankment Inspection and Evaluation Program

The data needed for inspection and evaluation shall consist of but not limited to the following

- The design memoranda to include principal design assumptions, stability, stress analysis, slopes stability, seepage and settlement analysis, consolidation, shear, permeability, compaction, classification test and contract plans and specification .
- Typical as-built plans, elevation and section.
- Selected as-built drawings of important project feature such as internal drainage, transition zones and reports of any special feature.
- Foundation data and geological features including boring profile, foundation mapping and subsurface exploration results.
- Location of borrow area and identification of embankment, filter, riprap and large stone source.
- Laboratory reports including
  1. As-built properties of foundation and embankment, such as shear strength, unit weight, water content and classification.
  2. Concrete properties such as physical, chemical, thermal properties, materials source and control procedures.
- Construction history records and sequences.
- All previous inspection reports and maintenance works.

If the existing data are insufficient or not available, it may be necessary to request supplemental investigation, analyses, or information to complete the evaluation. The information could involve additional visual inspections, measurement, foundation exploration and testing, materials testing, seismic information, hydrologic and hydraulics data.

2.4.3 Elements of Embankment Evaluation Process

To properly evaluate the stability and safety of an embankment dam, the following area should be reviewed

2.4.3.1 Embankment Zoning

The zoning geometry and properties of the materials placed in the zones should be reviewed to check

1) The structure design.
2) The types of internal features such as drainage device.

Desirable characteristics that these zones should have or provide are as follows

1- The core must meet the recommended limitation sated in previous sections

2- Transition zones must meet accepted filter criteria.

3-Seepage control features within the embankment Table 2.2 a- continue
should be sized adequately to contain all seepage flows.

4-Homogeneouse dams must have adequate internal drainage capability to ensure against seepage specially in case of absent of seepage control feature.

2.4.3.2 Seepage Control Measure

In the evaluation of seepage reduction or seepage control measure as they pertain to dam safety, one should review and evaluate the following

- Protective control measure which prevents seepage force from endangering the stability of the downstream slope.
- Filters and transition zones designed to prevent movement of soil particles that could clog drains or result in piping.
- Contacts of seepage control feature with foundation and abutment designed to prevent the occurrence of piping and /or hydro fracturing of embankment and/or foundation materials.

- Construction records for foundation shaping and treatment at contact between the impervious core and foundation.
- Measures such as compaction requirements, seepage collars, placement of special materials, or other similar features to prevent internal erosion from seepage at the interface with concrete structure.

2.4.3.3 Deformation Predicted or Recorded

The type, amount, and rate of deformation of an embankment, either vertical or horizontal movement, must be estimated during the design stage and should be recorded during the operation of the structure. For proposed embankments, the structure should generally be cambered to allow for the estimated settlement during the life of the structure.

For existing embankment, any evidence or records of unusual settlement, cracking, or movement should be reviewed to determine whether these conditions are detrimental to the continued safe operation of the structure. Field investigations may involve such items as surveying the structure. The embankment history, foundation conditions, hazard etc. are factor to be considered in determining field investigation needs.

2.4.3.4 Erosion Control Measure

Upstream and downstream slopes, the toe area, groin area of the abutments and area adjacent to concrete structure should be protected against excessive erosion from wave action, surface run off, and impinging currents. Inadequate erosion protection can result in slope instability.

The slope and toe protection should be reviewed to determine if the dam is adequately protected against erosive forces. If embankments materials consisting of stilly and sandy soil are being moved into the slope protection, measures must be taken to correct this condition before erosion becomes detrimental to the embankment.

2.4.3.5 Structural Stability Analysis:

The evaluation of stability of existing dam shall be based on design and construction information and records of performance. For existing embankment, the initial stability studies and analyses will normally be acceptable if they were performed by approved methodologies. Additional stability analysis should be performed if initial design analysis do not exist or are incomplete, if existing condition have deteriorated, if hazard potential of the project has increased, if the embankment has been subjected to loading conditions more severe than designed for, or if assumed design parameters can not satisfactory.

2.4.3.6 Soil Properties

Soil properties including strength and seepage parameters to be used as input data for stability analysis should be realistic and representative of the rang and variation that exist in the foundation, abutment, and embankment materials.

The selection of the proper input parameters and their correct use in stability analysis are generally of greater importance than the method of stability analysis used.

2.4.3.7 Embankment overtopping potential

Embankment dams should be evaluated for overtopping potential under the most extreme condition expected for which the dam is determined to be a hazard to life or property. The maximum reservoir elevation determined for the design flood and expected wave run up are condition should be considered. The free board should be reviewed to insure that it's height meets the limitation described in previous sections.

2.4.3.8 Potential of liquefaction

The phenomenon of liquefaction of loose saturated sand, gravel, or silt having a contractive structure may occur when such materials are subjected to shear deformation with high water pore pressure developing, resulting in a loss of resistance to deformation.

When embankment and their foundation are composed of loose sands, silts, or gravels, the analysis must be performed to determine: (a)if liquefaction potential exist and (b) whether such liquefied condition can lead to failure or excessive deformation of an embankment.

Generally the main characteristics of liquefiable soil are

- Percentage of finer than 0.005 <15%
- Liquid limit <35%
- Water content >0.9(liquid limit)
- Liquidity index <0.75

There is various simplified method available for evaluation of soil liquefaction potential based on empirical correlation between in situ behavior of soil and standard penetration resistance. Appendix A illustrates the empirical method which developed by Seed .H.B

2.4.3.9 Compaction requirement

Generally the density, permeability, compressibility, and strength of impervious fill materials are dependent upon water content at the time of compaction, for latter reason compaction and compaction control may consider a very important process in the embankment construction.

The quality of compaction controlled by comparing the field densities and water content with design water content and densities which must be selected depends on many factors such as

- Borrow area water content
- Climatic Condition
- The influence on construction cost
- The type and height of embankment
- Settlement of compacted materials on saturation.

The normally assumption is field densities will not exceed the maximum densities obtained from the standard compaction test nor be less than 95 percent of the maximum densities derived from this test, and the placement water content must range between 2% dry – 3% wet of optimum water content.

To evaluate the quality of compaction of the embankment material many field density tests must be performed and comparing the results with design specification.

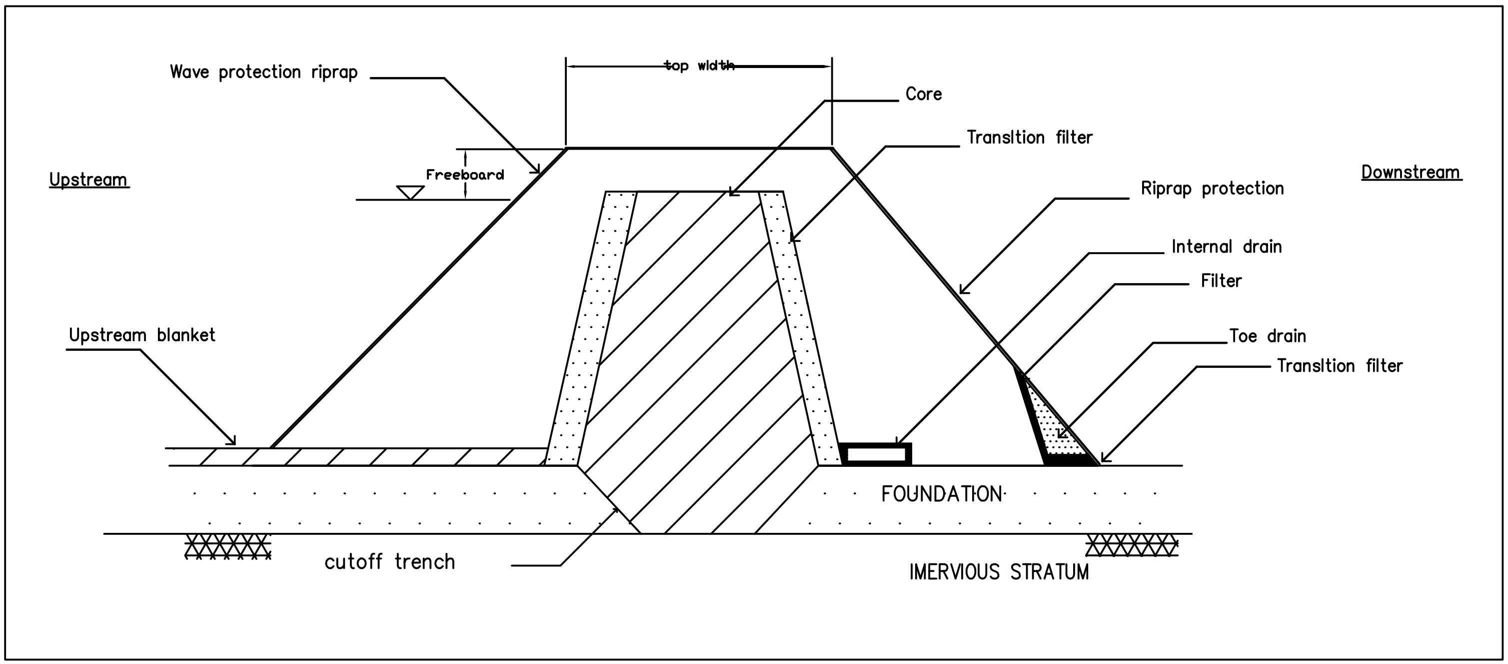


Figure 2.1 Parts of earth dam
Note: not all of the above ordinarily would be incorporated in any dam

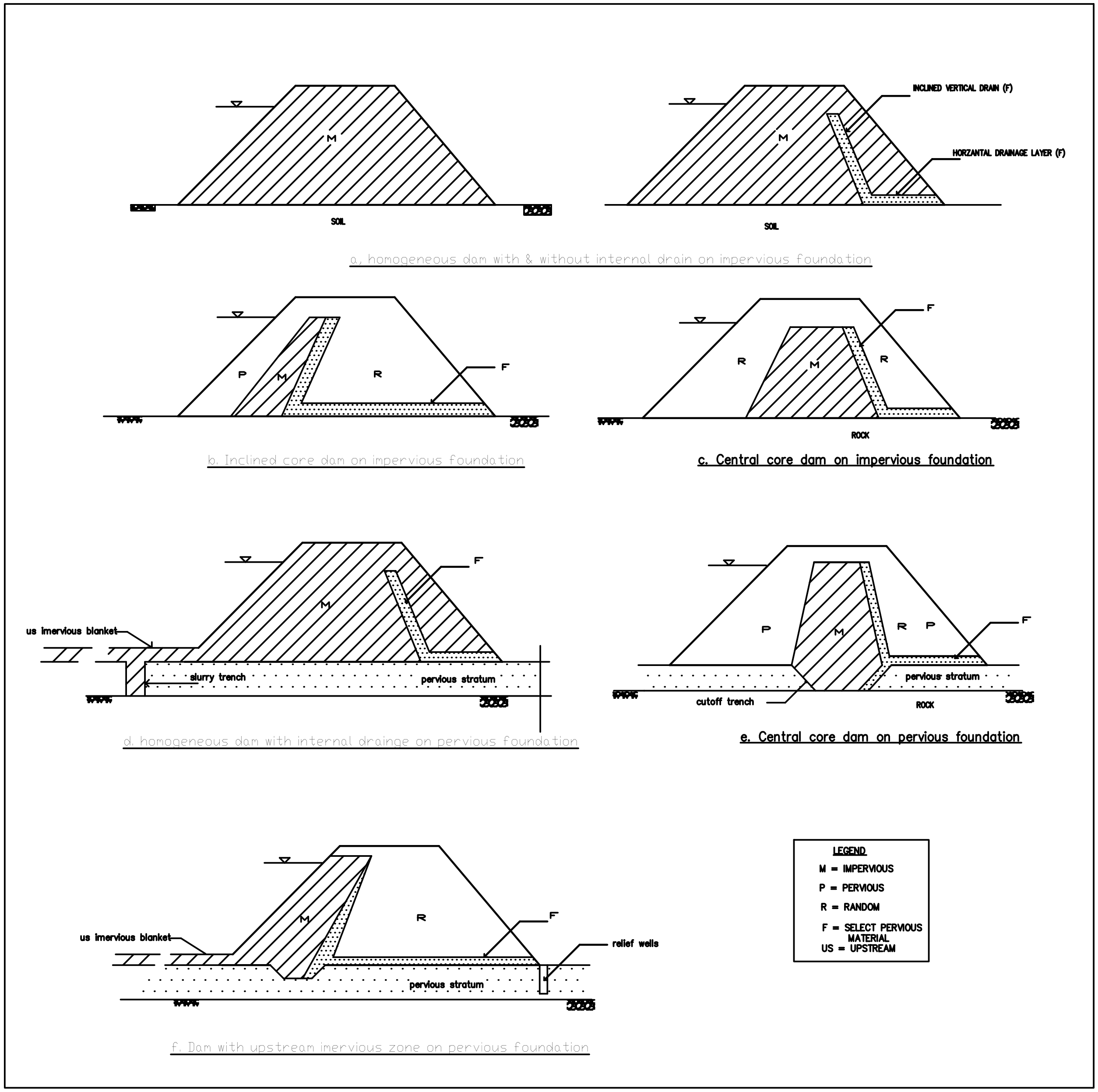


Figure 2.2 Types of earth dam section

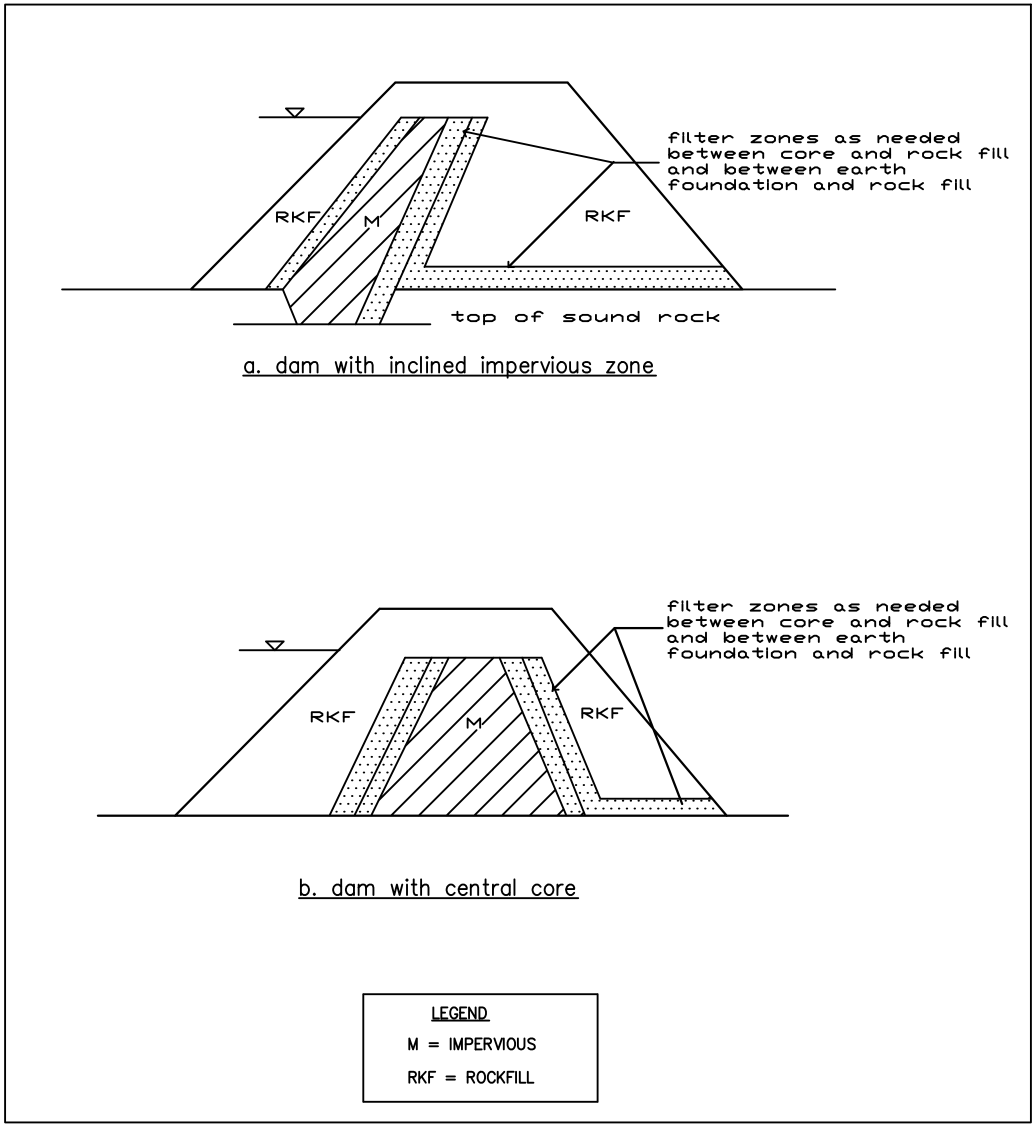


Figure 2.3 Two types of rock-fill dams

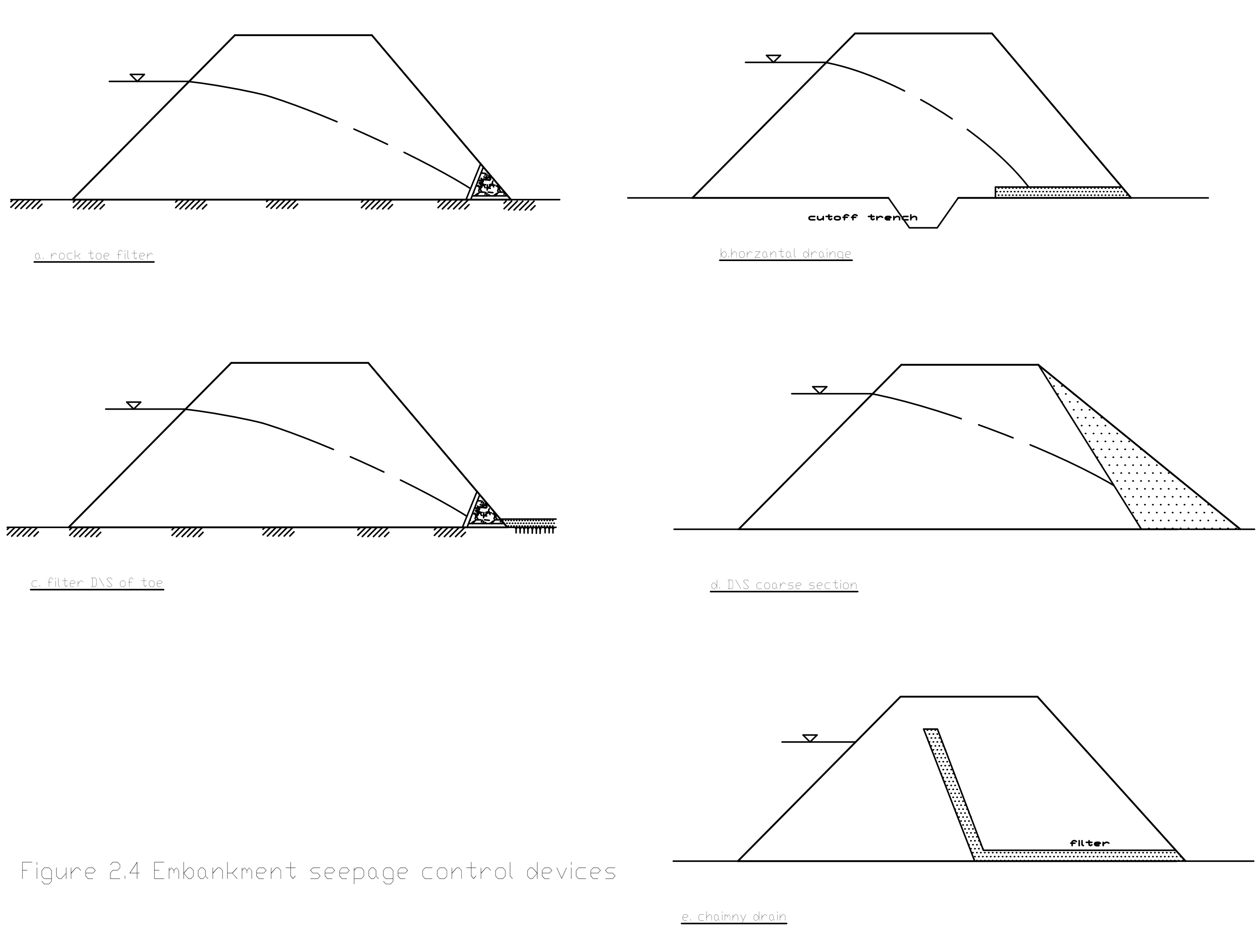


Figure 2.4 Embankment seepage control devices

CHAPTER (3)
DETAILS OF STUDY PROGRAM

3.1 Introduction

The main purpose of this study is to evaluate an existing embankment that was constructed few years ago. This chapter describes the present condition of the specific embankment. The chapter reviews the available data obtained from the technical reports prepared by embankment designer. This chapter is outlines the program planned and executed by Building and Road Research Institute BRRI, University of Khartoum to check and complete the data to be used in the evaluation process.

3.2 Review of Available Data for Existing Embankment

3.2.1 General

The available data and information was obtained from the designer's technical report and personal interview with personnel who were in chief of construction supervision. Generally, the available data was limited and inadequate and this represents the main difficulty faced the embankment evaluation process. The following points are a general examination of available information and data

3.2.2 Type and Purpose of Embankment

The embankment under focus is a homogeneous earth embankment constructed to protect the urban area from river flooding. Figure 3.1 and Table 3.1 illustrate the cross-section dimension and properties of the constructed earth embankment as described by designer

Table 3.1 Main feature of embankment section (specified by designer)

Section title	Description
Embankment height	6.5m above the natural ground level (375.0) reduce level 381.5m
Top width	10m
Slopes	Upstream slope 1:2.5 (V:H) Downstream slope 1:2 (V:H)
Free board	-1.5m above maximum flood level
Cut off trench dimension	Depth below embankment base equal 1.5m base width 3.0m slopes 1:1 (V:H)
Embankment length	1050m
Erosion control	- Upstream pitching consist of 0.5m rock layer placed on supporting gravel layer ranges between 0.15-0.25m - Downstream pitching consist of 0.4m rock layer placed on supporting gravel layer (ranges between 0.45-0.25m) - Filter layer (sand) 0.15-0.2m thickness placed under slopes pitching.
Seepage control	- Horizontal drainage filter, 0.5m thickness and 4.5m length started from downstream drain trench -Horizontal filter joined with longitudinal downstream trench.

3.2.3 Embankment Materials

The homogeneous impervious clay soil was specified by the designers for use in the embankment construction, the selected soil has the following properties

- dry unit weight = 16 kN/m^3
- cohesion = 5 kN/m^2
- internal friction angle = 17°

The actual soil materials used for construction were borrowed from Jebel Torriya in north- western of Omdurman and there was no evidence to prove that further classification tests were carried out in borrows area on these materials. Moreover, there is no information about other materials which have been specified by the designers for used in the construction of the embankment, such as sand and gravel to be used in filters, and rocks.

3.2.4 Hydraulic Information

The maximum full supply level of water from river was assumed as 380m about 1.5m below the maximum crest level, and the minimum river level was assumed 373m about 2m below the embankment base level.

3.2.5 Foundation Condition

The designer's technical report does not give any information about foundation materials i.e. types, shear strength properties, and substratum profile.

3.2.6 Slope Stability

A computer software program was used by the designers for the analysis of slope stability of the embankment and the minimum factor of safety against slope slide under minimum and maximum water level for the steady state are listed in Table 3.2 below `

Table 3.2 Minimum factor of safety against slope slide calculated by designer

Load condition	Up stream slope	Down stream
Steady state seepage	1.54	2.1
Rapid drawdown	1.29	-

The limit equilibrium method was used in the analysis of embankment slope stability

3.3 Details of Field Data Required for Study

3.3.1 Source of Data

The inadequacy of available data and information on embankment lead to performing an inspection program, this program contain

- General Field Assessment of existing embankment Condition
- Geotechnical investigation including
  a. Field activities.
  b. Laboratory test.

3.3.2 Field Assessment of Embankment

The main objectives of field assessment carried out in this study were as follow

(i) To check the geometry and dimension of the constructed embankment section for the purpose of comparison with those prepared by the designer.

(ii) To find out any settlement evidence which appeared in the embankment body.

(iii) To observe erosion action that can disserve the embankment.

(iv) To check the slope protection condition.

(v) To verify stability of slope.

(vi) To check the situation of apparent parts of seepage device.

The field assessment comprised of the following tasks

- Site vists for embankment inspection
- Survey works

The results of site inspection and survey works are listed later in chapter 4.

3.3.3 Geotechnical Investigation on Embankment Evaluation

3.3.3.1 Objectives

The main objective of geotechnical investigation carried out by Building and Road Research Institute (BRRI) and referred to in the previous section are

(i) Checking the foundation condition, through exploration of the subsurface soil and its depth, and determine the engineering properties of foundation soils in order to evaluate their suitability and capability for supporting the embankment.

(ii) Obtain the soil parameters which may be used in stability analysis.

(iii) Checking the suitability of the soil used for construction and making comparison with that soil specified by designer.

(iv) Checking the quality of filling process through determining compaction characteristics of the compacted fill material.

The geotechnical investigation comprised the following activities

i. Field works which include
    - Drilling of deep borehole
    - Performing static cone penetration test (CPT)
    - Digging shallow test pits for sampling and performing field density test

ii. Laboratory Testing which includes performing the following tests
    - Sieve analysis and hydrometer
    - Atterberg limits
    - Specific gravity
    - Shear test [consolidated – undrained tri- axial test]
    - One dimensional consolidation
    - Compaction
    - Permeability
    - Dispersion test (Pinhole test and Double hydrometer test).

Figure 3.2, 3.3a and 3.3b show the location of borehole and CPT tests and the location of field density test and inspection pits. Brief description of field activities and laboratory tests are given in the following section

3.3.3.2Details of Field Activities

(i) Borehole drilling

Drilling of the borehole was carried out by conventional auger method of penetration through soil strata to the depth between (15-18) m deep. Both disturbed and undisturbed soil samples were collected from several borehole and test pit depths to inspection and identification of the embankment and its foundation materials.

The soil profiles as made on the basis of borehole logs are shown in Appendix B.

(ii) Static cone penetration test (CPT)

A 200 kN capacity mechanical machine equipped with an adhesion friction jacket cone was used in CPT test. These tests were started from the embankment crest level and extended to 19m depth. The CPT data obtained from the results at 6 test points along the embankment are shown in appendix C.

(iii) Test pits and field density test

In order to perform the laboratory test (classification, dispersion, standard compaction and permeability), the disturbed samples were obtained from shallow pits dug along the embankment crest and slopes as shown in Figures 3.3a and 3.3b. Moreover, field density tests were performed at bottom of test pits to determine the field dry density and moisture content of the embankment soil using the sand replacement test method.

Table 3.3 below show the numbers and locations of the field density test.

Table 3.3 In-situ density test (sand replacement method) number and location

Number of test	location	Test Designation as Figure 3.3a	Remarks
11	Down stream	TP1D, TP2D… to TP11D	Performed at D/S slope
8	Crest	TP12 Top… to TP19 Top	Performed at depth about 10 cm
8	Crest	TP20 Top… to TP27 Top	Performed at depth about 50cm
6	Upstream	TP1U, TP2U… to TP6U	Performed at U/S slope

Table 3.4a, shows the results of field dry density tests. Table 3.4 b shows the results of field density tests obtained from the undisturbed soil samples collected from three borehole (at different depths)which were used for determining some properties in the laboratory.

3.3.3.3 Laboratory Testing Details

i. Grain Size Distribution

The grain size distribution test was carried out to determine the type and gradation of embankment and foundation material. This test was made on 30 soil samples (12 samples from the embankment body and 18 samples from the foundation) collected at 1.5m intervals for the borehole depths ranging between 1.5m and 15m below embankment crest level.

Also, hydrometer tests were performed on nine fine grained soil samples, (1.5, 3.0 and 4.5m depth from each borehole), to determine the percentages of the silt and clay fractions of the soil.

The grain size distribution curves obtained by the sieving and hydrometer tests or sieving only are shown in Appendix D, and the results of these tests are summarized in section 4.2.

ii. Atterberg Limits

The liquid and plastic limits tests were performed on samples collected from various borehole depth and results obtained are summarized in Table 3.5.

iii. Specific Gravity

Specific gravity tests were performed on nine samples taken from each borehole, and the results are summarized in Table 3.6.

iv. Consolidated-Undrained Triaxial Test (CU)

In order to check the embankment stability and foundation bearing capacity consolidated-undrained tests were performed on nine undisturbed samples. The effective shear parameters (internal friction angle (ǿ) and cohesion (ć)) were determined from the results of the CU test, and Table 3.7 summarize of the results.

In addition, the CU triaxial test results were plotted as Mohr's circle and shear failure envelopes as shown in Appendix E.

v. One-dimensional Consolidation Test

Consolidation tests were performed on nine samples collected from the foundation soil at borehole depths ranging between 6-12m below the embankment crest level. The results of these tests were listed in Table 3.8, and the consolidation results are plotted in Appendix F.

vi. Permeability Test

The falling head test procedure was followed in the laboratory to determine the permeability of embankment material, and the values of only two tests are obtained, in which the coefficient of permeability (K) is 4.5E-10 m/s and 1.93E-11 ms.

vii. Compaction Test

To determine the dry density- moisture content relationship for the soil material used in embankment construction, the ordinary compaction test (2.5 Kg rammer or standard proctor method) was performed on 6 samples collected from test pits.

Table 3.9 gives the summary of the compaction tests results and Appendix G shows the compaction tests curves obtained for tested samples.

viii. Dispersion Tests(Double Hydrometer Test and Pinhole Test)

To evaluate the dispersive behavior of the material used for construction, the double hydrometer test and pinhole test performed on samples collected from 6 different locations along the main axis. The results of double hydrometer test are shown in Appendix H and results of pinhole tests are given in Table 3.10.

Table 3.4 a: Results of field density test(sand replacement)

soil identification	test location	field dry density(FFD)gm/cm3	field moisture content(FMC)%	degree of compaction%	FMC-OMC*
TD1D	D/S	1.58	11.3	96.9	-7.9
TD2D	D/S	1.05	10.4	64.4	-8.8
TD3D	D/S	1.33	9.6	81.6	-9.6
TD4D	D/S	1.66	8.4	101.8	-10.8
TD5D	D/S	1.71	6.4	104.9	-12.8
TD6D	D/S	1.68	8.7	103.1	-10.5
TD7D	D/S	1.61	7.8	98.8	-11.4
TD8D	D/S	1.2	14.6	73.6	-4.6
TD9D	D/S	1.36	12	83.4	-7.2
TD10D	D/S	1.65	11.5	101.2	-7.7
TD11D	D/S	1.66	7.8	101.8	-11.4
TP12TOP	CREST	1.57	6.6	96.3	-12.6
TP13TOP	CREST	1.49	8.8	91.4	-10.4
TP14TOP	CREST	1.53	8.7	93.9	-10.5
TP15TOP	CREST	1.52	6.4	93.3	-12.8
TP16TOP	CREST	1.56	10.5	95.7	-8.7
TP17TOP	CREST	1.45	9.3	89.0	-9.9
TP18TOP	CREST	1.48	13.2	90.8	-6.0
TP19TOP	CREST	1.73	5.9	106.1	-13.3
TP20TOP	CREST	1.77	9.8	108.6	-9.4

TP21TOP	CREST	1.5	11.9	92.0	-7.3
TP22TOP	CREST	1.53	11	93.9	-8.2
TP23TOP	CREST	1.37	12.2	84.0	-7.0
TP24TOP	CREST	1.73	6.3	106.1	-12.9
TP25TOP	CREST	1.32	16.4	81.0	-2.8
TP26TOP	CREST	1.28	17.8	78.5	-1.4
TP27TOP	CREST	1.38	11.9	84.7	-7.3
TP1U	U/S	1.71	22.9	104.9	3.7
TP2U	U/S	1.34	22.8	82.2	3.6
TP3U	U/S	1.43	20.6	87.7	1.4
TP4U	U/S	1.34	22.1	82.2	2.9
TP5U	U/S	1.24	25.8	76.1	6.6
TP6U	U/S	1.19	25.3	73.0	6.1

***Results of standard comaction :**

optimum moisture content OMC=19.28%

maximum dry density MDD=1.63 gm/cm3

degree of compaction=FDD/MDD

Table 3.4 b: Results of field density test(performed in borehole)

Borehole NO.	test depth m	field dry density(FFD)gm/cm3	field moisture content(FMC)%	degree of compaction%	FMC-OMC*
1	1.5	1.54	15.1	94.5	-4.13
1	3	1.36	18.3	83.4	-0.93
1	4.5	1.81	17.2	111.0	-2.03
1	6	1.53	11.5	93.9	-7.73
2	1.5	1.37	22.2	84.0	2.97
2	3	1.55	15	95.1	-4.23
2	4.5	1.79	14.9	109.8	-4.33
2	6	1.64	16.8	100.6	-2.43
3	1.5	1.53	18.6	93.9	-0.63
3	3	1.58	13.4	96.9	-5.83
3	4.5	1.57	14.7	96.3	-4.53
3	6	1.72	14.6	105.5	-4.63

Table 3.5: Summary of Atterberg limits

Sample ID		Atterberg limits		
BH No	**Depth**	**L.L (%)**	**P.L (%)**	**PI (%)**
1	**1**	52	22	30
	3	59	24	35
	4.5	43	16	27
	6	51	21	31
	7	55	20	35
	9	36	27	9
	10	49	29	20
	12	43	25	18
	13	47	27	20
	15	-	-	NP
2	**1.5**	53	25	28
	3	48	22	26
	4.5	43	17	26
	6	47	18	29
	7	47	19	28
	9	44	26	18
	10	46	26	20
	12	51	24	27
	13.5	-	-	NP
	15	-	-	NP
3	**1.5**	54	23	31
	3	52	23	29
	4	61	23	38
	6	47	22	25
	7	46	18	28
	9	54	25	29
	10.5	45	24	21
	12	49	24	25
	13.5	33	19	14
	15	34	16	18

Table 3.6 : Results of specific gravity tests

Borehole No.		Average specific gravity
1	depth 6m	2.69
	depth 9m	2.76
	depth 12m	2.69
2	depth 6m	2.78
	depth 7.5m	2.7
	depth 10.5m	2.72
3	depth 7.5m	2.77
	depth 9m	2.77
	depth 10.5m	2.72

Table 3.7: Results of Consolidated-Undrained shear tests

Borehole No.		Internal friction angle(degree)	Cohesion(kN/m2)
1	depth 1.5m	24	16
	depth 4.5m	21	10
	depth 7.5m	25	20
2	depth 3m	22	16
	depth 6m	21	28
	depth 9m	38	0
	depth 10.5m	38	0
3	depth 3m	27	18
	depth 7.5m	26	9

Table 3.8: Results of consolidation tests

Borehole No.		Initial void ratio(e0)	Preconsolidation pressure Pc(kN/m2)	compression index (Cc)	swelling index (Cs)
1	depth 6m	0.7	186	0.2224	0.0543
	depth 9m	1.275	280	0.4504	0.05
	depth 12m	1.357	105	0.1941	0.0189
2	depth 6m	0.515	1060	0.722	0.0345
	depth 7.5m	0.603	100	0.217	0.0465
	depth 10.5m	0.972	190	0.2795	0.0288
3	depth 7.5m	0.602	175	0.168	0.0668
	depth 9m	0.946	1060	0.722	0.0345
	depth 10.5m	1.086	100	0.1241	0.0219

Table 3.9 : Results of standard compaction test

Sample serial No.	Max dry density (gm/cm3)	Optimum moisture content OMC%
1	1.65	17.5
2	1.59	20.6
3	1.77	15.8
4	1.56	22.3
5	1.61	20
6	1.61	19.5
average	1.63	19.28

Table 3.10 : Results of dispersion test (pinhole tests)

Sample Id.	Dispersion category	Degree of dispersion
1	ND2	Non dispersive
2	ND2	Non dispersive
3	ND4	Potentially dispersive
4	ND4	Potentially dispersive
5	ND3	Potentially dispersive
6	ND2	Non dispersive

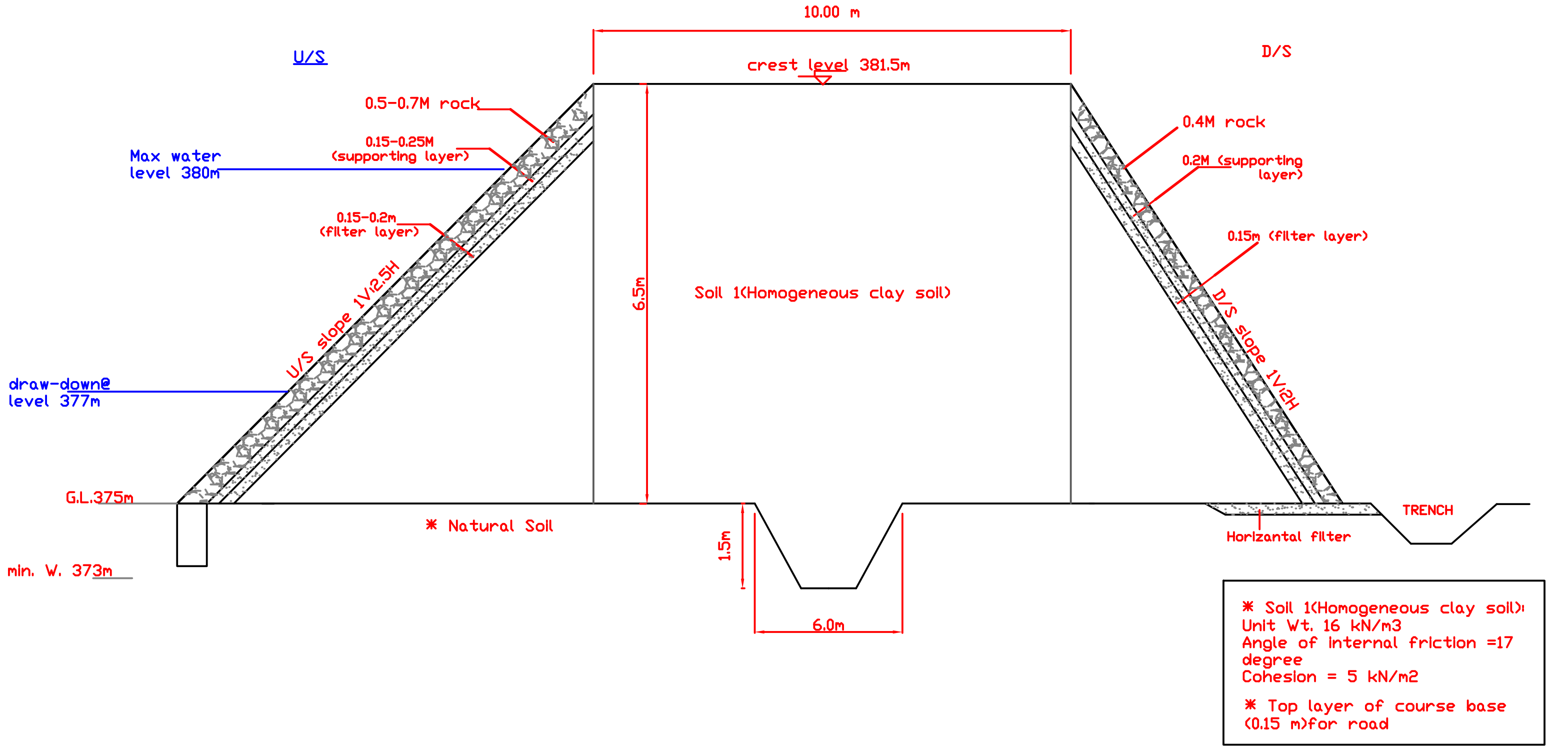


Figure 3.1: Embankment section dimension & properties as specifed by design

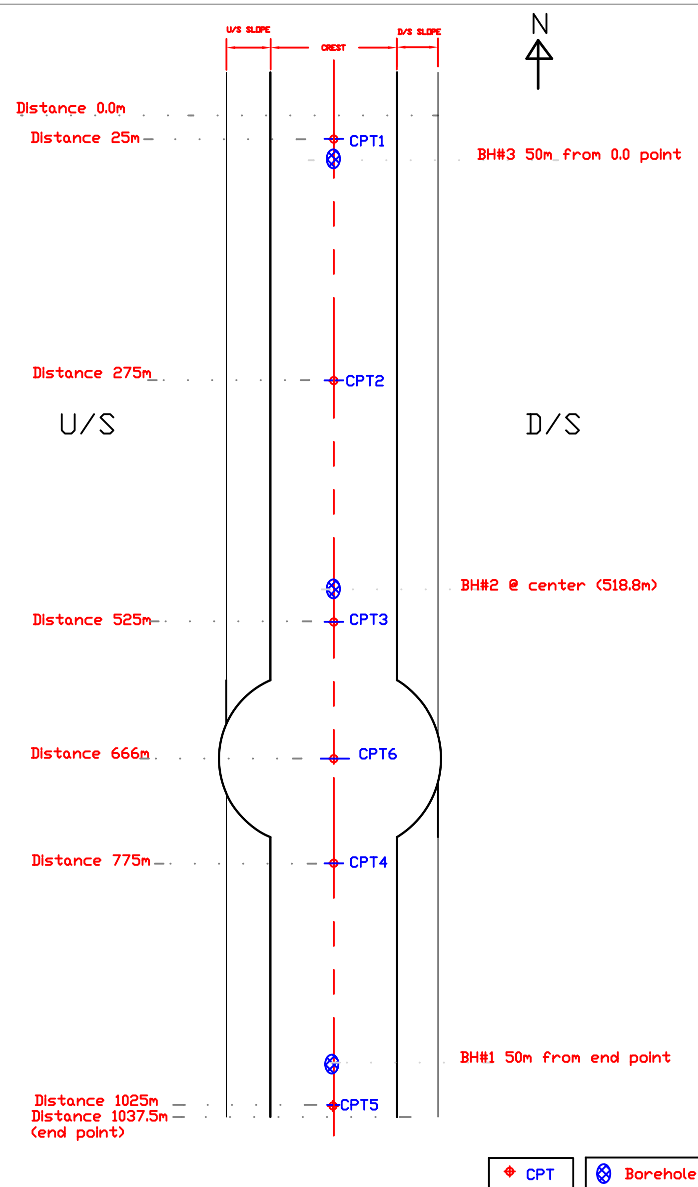


Figure 3.2 :Embankment plan
Position of CPT & borehole tests

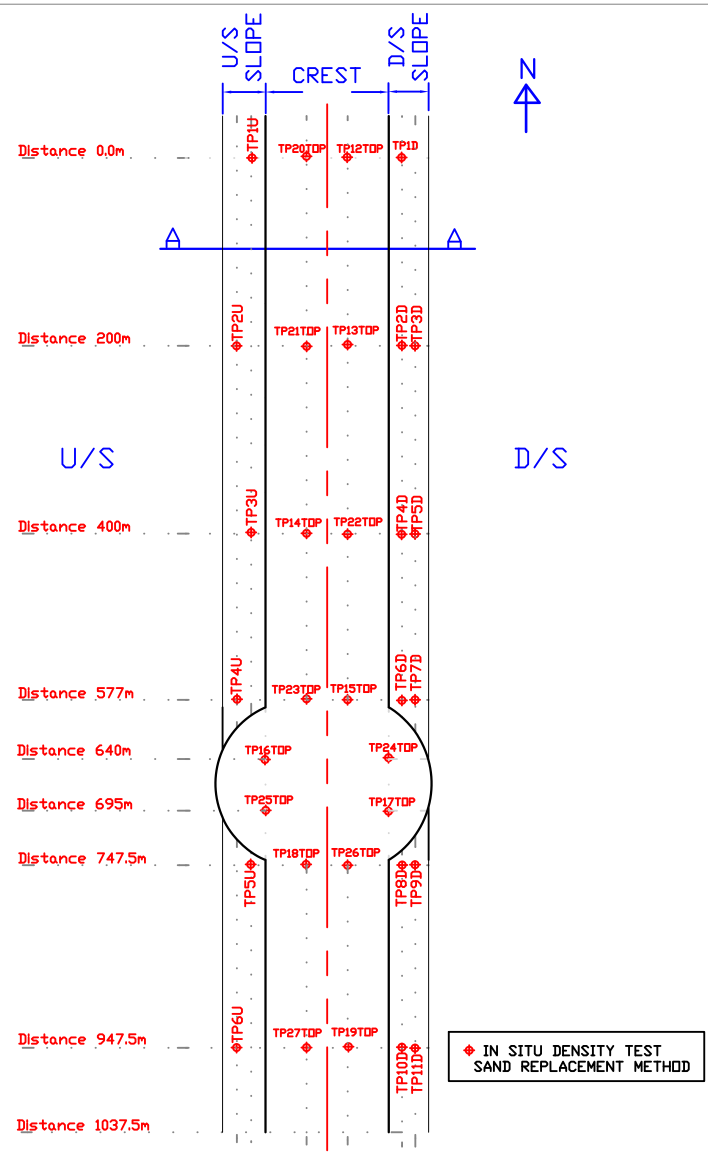


Figure 3.3a: location of field density tests and pits

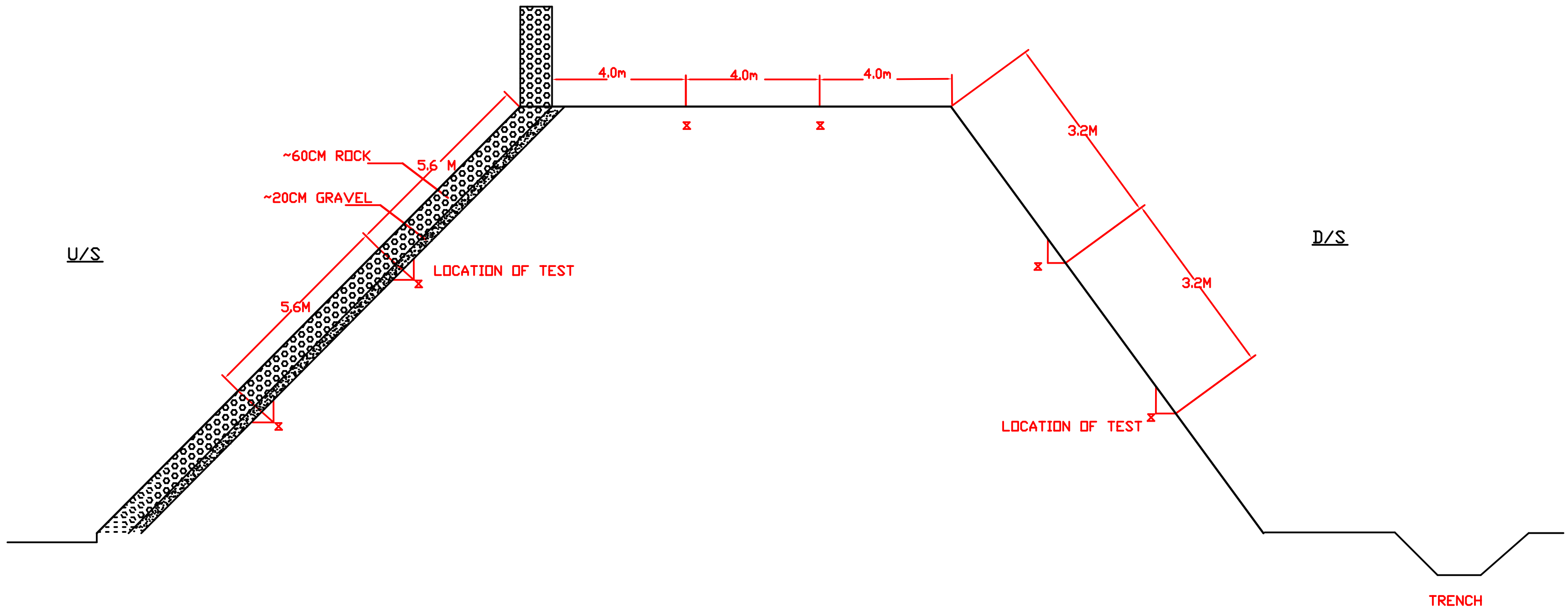


Figure 3.3 b : Embankment X-section
and location of field density tests

CHAPTER 4
ANALYSIS AND DISCUSSION OF STUDY RESULTS

4.1 Introduction

After the compilation of the data and information pertaining to the existing embankment undertaken through the field assessment and laboratory tests, the second step would be the analysis of this data and results to compare the details of existing embankment to those specified in design, and to evaluate the stability, safety and the expected future performance of the embankment.

The important evaluation aspects to be considered in the analysis include the following

- Evaluation of the embankment section and geometry.
- Evaluation of the quality of the embankment materials and placement defects.
- Check of the slopes and foundation stability.
- Evaluation of the seepage control.
- Evaluation of the erosion control.
- Looking for earthquake and liquefaction aspects.

Each of these aspects shall be discussed separately in the following sections

4.2 Analysis of Field Assessment and Geotechnical Investigation Results

From previous presentation of available data and geotechnical investigation results there is conclusion which describing the present situation (geometry, material, foundation, and features) of existing embankment here are the details of this conclusion obtained through the inspection works, and obviously there are many differences between data obtained from designers specification and data obtained from inspection works.

The following points and phases are illustrating the final results concluded from combination of design report and inspection work described in chapter 3, also plates 1, 2, 3 and 4 Appendix I show the general view of the embankment component.

i) The Embankment Section and Feature

Figure 4.1 and Table 4.1 below show the existing section dimension and feature of the embankment

Table 4.1: The main feature of existing embankment section

Section title	description	Source of data
Embankment length	1037.5m	Field measure
Embankment height	- Crest reduces level ranging from 382.11-382.22m (average value382.19m).	Surveying works
Top width	12m	Field measure
Slopes	-Upstream slope 1:2.9 (V:H) -Downstream slope 1:1.9 (V:H)	Surveying works
Free board	2.19m above the maximum flood level	Surveying works
Cutoff trench dimension	-No new data more than data specified by designer	Design report
Erosion control	- Upstream pitching consist to rock fragment thickness - No any erosion control and protection performed on downstream of crest	-upstream test pits - site visit
Seepage control	- No evidence prove the existence of horizontal drainage filter	-Field excavation

	- downstream trench	-Design report and site visit
Additional component	-Circular shaped portion as shown in Figure 3.2 and photograph, with diameter about 162.5m and started from 585.5m from North edge of embankment (see plate 5 Appendix I) - Concrete stairs in upstream slopes - Wave wall on upstream slope	Site visit

ii) Embankment Materials

Generally, the material used for the construction of the existing embankment was a clay soil. A table 4.2 shows the general description of this soil and its index and engineering properties.

Table 4.2 The material properties of the existing embankment

Title	Description
Type	- According to the unified soil classification system the type of embankment soil is CH and CL [inorganic clay of low to medium plasticity] - Moreover there are pockets of clayey slit (SC)
Index properties i.Component	- The percentage of components:- Clay ranged between 6% -22% Silt ranged between 24 %-54% Sand ranged between 34%-54% Gravel ranged between 1%- 7% -Fines content ranged between 45% to 71 %

<table>
<tr><td>ii.Plasticity

iii.Specific gravity</td><td colspan="3">Plastic limit 16% - 54%
Liquid limit 43% - 61%
Plasticity 25% - 38%
Between 2.69 to 2.78</td></tr>
<tr><td>iv. Field density and moisture content</td><td colspan="3">-Field density ranged between 1.36gm/cm^3 to 1.81 gm/cm^3 with average value 1.58gm/cm^3
- U/S moisture content 20.6% - 25.8%
-D/S moisture content 6.4% to 12%
-Crest moisture content 5.9% to 17.8%</td></tr>
<tr><td>Engineering properties
i. permeability</td><td colspan="3">Coefficient o f permeability K= 4.5E-10 m/s to 1.93E-11 m/s</td></tr>
<tr><td rowspan="3">ii. effective shear strength parameter</td><td>parameter</td><td>ǿ degree</td><td>ć(kN/m^2)</td></tr>
<tr><td>range</td><td>21-27</td><td>10-28</td></tr>
<tr><td>average</td><td>23</td><td>17.6</td></tr>
</table>

iii) Foundation condition

Foundation classification and description was done through the results of field works (borehole and CPT tests) and laboratory test with aids of unified soil classification system and developed classification system prepared by BRRI to use CPT results to classify the Sudanese soil.

These results shown up a 5.5m to 8.5m of dark brown silty clay layer located directly under embankment base, this layer is low to high plasticity soil and becoming dark gray with depth. The clay layer rests on a dark gray stratum of very loose to dense, medium to coarse grained silty sand of thickness about 3m and extends to depth about 10m below the original ground level.

The sand layer is underlined by hard Nubian formation, i.e. alternating layer of hard stone and mudstone extending down to the maximum depth of 11.5m below the original ground level reached in this investigation.

Ground water level was located at depth of 9.5m below embankment crest level (3.0m below original ground level). Figure 4.1 show the general foundation soil section as described above.

4.3 Evaluation of the Embankment Section and Geometry

As stated above (section 4.2),Figure 4.1 shows the cross-section of the embankment, and also show the variation in executed section details from those proposed in the design drawings.

The main observations and comments which relate to the section and geometry of the embankment are given hereunder

i. The total measured length of the embankment is 1037.5m while the length given in the design report was a little pit longer (1050m).

ii. The designer specified a 6.5 m height of the embankment in the drawings at crest reduced level of 381.5m, but the actual crest reduced level is 382.19m (average value). This means that the constructed level is higher than the level shown on the drawings by approximately 0.7 m.

iii. The actual average width at the top of embankment is 12m measured at crest level, and the design drawing specified 10 m width, i.e. there is 2 m increase in the proposed width.

For compare the actual and design width the suggestion of the U.S Bureau of Reclamation in which b (top width) given by the expression

$b = 1.65\,(Z + 1.5)^{2/3}$ where $Z \equiv$ embankment height.

One finds that the value of above equation give 5.88m “say 6m” for top width. That means there is increase in actual design top width by two times the above suggested value, whereas no public traffic will be allowed to pass on the embankment crest.

The actual embankment crest width is overestimated in the design as from practical viewpoint; the purpose of the embankment does not needs such great crest width.

iv. The survey work confirmed that the average upstream slope is IV: 2.9H and the average downstream slope is 1V: 1.9H thus, the upstream slope is flatter whereas the downstream slope is slightly steeper than the respectively recommended design slopes of the embankment.

Table 2.5 Chapter 2 shows the side slopes for an earth dam which according to Terzaghi, the suitable slopes for homogeneous clay dam is 1V: 2.5 H for upstream, whereas 1V: 2H for downstream for height less than 15 m. The slopes proposed by the designer are in corresponding Terzaghi recommendation but the actual slopes not corresponding the Terzaghi's value. The stability of actual slopes was checked and summarized in Section 4.5.

v. The actual freeboard is 2.19 m with an increase of 0.69m compared to the 1.5 m freeboard proposed in the design that excluding the possible reduction in the free board due to the post-construction settlement.

The U.S.B.R suggested the minimum free board 2 m (Table 2.4) over maximum flood level. Fortunately the constructed freeboard (construction defect) meets the U.S.B.R recommendation but the proposed design is not.

By calculating the minimum freeboard using the method described in Appendix A the wave heights equal 0.33 m, the calculation were based on an assumed that the fetch of 1km (the distance equal 1 km between the embankment and land sourrounding the body of water) and wind velocity of 45 km/ hr. The design freeboard is sufficient against overtopping potential by the wave action during high flood level.

vi. There are some additional components not observed in the original design drawings of the embankment, these include large circular shaped portion and the wave wall component. No mention was however made in the design report on the function of such a major embankment component, or its effect on the stability and safety of the whole embankment. This circular portion has partially failed in the downstream face(plate 6 Appendix I) possibly due to the defect in the drainage of rain water (see Plate 7 Appendix I which shows the difference in level between the circular portion crest and drainage hole located in the slopes retaining walls for drainage purpose). Also there is no mention of the wave wall in the end of upstream slope in the design report, but this part has not suffered from any kind of distress.

4.4 Evaluation of the Embankment Materials and Fill Placement Defects

From classification of embankment materials, described in the previous chapter, it is clear that the embankment was constructed by heterogeneous material (CH and CI) with presence of pockets of (SC). The design specified that homogeneous clay is to be used in the embankment construction, Moreover there was random soil placed during construction and the presence of this random soil in the embankment body may be seen in Plate 8 Appendix I.

The soil used in construction of the embankment is practically impermeable as indicated from permeability test results summarized in Table 4.2.

On the basis of description of embankment materials given above and in the previous chapter, the main properties of embankment material the following points can be summarized as follow

(i) Overall soil used in the construction doesn't meet the design requirements, because no tests were carried out in the borrow area for selecting the suitable material.

(ii) The embankment material is practically impermeable and this is quite satisfied and useful specially for seepage purpose.

(iii) The soil used in construction has a high plasticity (CH soil) and this may have a serious instability effect specially in case of sequences of wetting and drying.

(iv) The soil material is not workable(during construction process) due to the presence of CH and CL soil and this due to high plasticity

(v) The pockets of (SC) soil and random soil indicate significant placement defects, and lack of quality control testing during construction.

(vi) According to double hydrometer and pinhole test results the soil used in embankment construction may be classified as a non-dispersive material.

(vii) Compaction requirement , by comparing the results of field density test which were performed on existing embankment with result of standard compaction test (each test results summarized in Table 3.4a and b , Table 3.9 respectively), one may note that

- Optimum moisture content (OMC %) = 19.28% , and maximum dry density(MDD gm/cm^3) = 1.628 gm/cm^3
- Just 44% of test results have degree of compaction over 95% (95% which is normally represents minimum limit for acceptance), and 56% of test results have a degree of compaction less than 95%.
- The whole of the embankment is dry of optimum moisture content ,the actual moisture content of placed soil is very low with average difference of 0.7 to 13.4 below OMC specially in the downstream and the top of the embankment.
- The upstream portion is wet; the moisture content is higher with an average difference of 1.3 to 6.5 above OMC value.

- The dry field densities of fill are not reaching the desirable design of compaction (95%).
- The moisture content differ significantly from point to other when compared with normally acceptable difference of 2% dry to 3% wet of optimum water content.

From above comparison and evaluation it may be concluded that the embankment fill materials was not satisfactory. This may be attributed to lack quality control testing before and during the construction

Comparing the soil used in the embankment construction with those recommended by Indian standard (Table 2.3), it could not be classified as homogeneous but is acceptable according to this standard (see Table 4.3).

Table 4.3 General characteristics of the soil of various groups classified by I.S.C system and U.S.C system

Soil group	**Permeability**	**Compressibility**	**Shear strength**	**Workability**
GC	Impervious	Very low	Good to fair	Good
CL and CI	Impervious	Medium	Fair	Fair
CH	Impervious	High	Poor	Poor
SC	Impervious	Low	Good to fair	Good

4.5 Stability of Slope and Foundation

4.5.1 Stability of Upstream and Downstream Slope

A slope stability analysis was performed using the computer program (MZ Stable Geotechnical Software Version 9.03) using circular arc surface Modified Bishop method to calculate minimum factor of safety against failure. The result of analysis is summarized in Table 4.4 and compared with U.S Corps of Engineer recommended listed in Table 2.6. Two sets of soil parameters (obtained from CU tests) were used in the analysis, minimum and average value of internal friction angle (ϕ') and effective cohesion (ć) as shown in Table 4.5.

Figure 4.2a and b and Figure 4.3a and b show the results of analysis of two assumed loading condition i.e. the steady seepage and rapid drawdown including the circular slip surface in each case.

The approximate method described in Appendix A was used also for calculating the factor of safety or sloughing of upstream slope during sudden drawdown, and the factor of safety obtained is 2.87 , based on the average value of soil parameters the required value is F.S =2.0, therefore the upstream slope of embankment is safe. The analysis showed that the upstream and downstream slopes are stable against failure during two loading condition (rapid drawdown and steady state seepage). The calculated values exceeded the desired safety factors specified by the U.S Corps of Engineer except in case of minimum parameter condition during steady seepage conditions (calculated F.S was 1.208 versus required value of 1.5). This has no practical significance of slope stability because the embankment is unlikely to reach the condition of complete saturation during flood season.

Table 4.4 Summary of results of slope stability analysis and comparison with U.S corps of Engineer recommended value

Analysis Condition		**Minimum safety factor**			
		Downstream slope		**Upstream slope**	
		Actual F .S	**U.S.C.E** F. S	**Actual** F.S	**U.S.C.E** F .S
Steady seepage	Min properties	1.208	1.5	4.008	-
	Average properties	1.965	1.5	5.108	-
Rapid drawdown	*Min properties*	1.393	1.2	1.983	1.2
	Average properties	2.167	1.2	2.95	1.2

Table 4.5 Summary of Soil parameter used in slope stability analysis

	Soil 1 Embankment material			**Soil 2 Foundation material**		
	Cohesion C' (kn/m^2)	Friction ϕ' degree	Unit density γ (kn/m^3)	Cohesion C' (kN/m^2)	Friction ϕ' degree	Unit density γ (kN/m^3)
Minimum Value	10	21	16.2	0	25	18.0
Average Value	17.6	23	18.3	9.0	31.75	19

4.5.2 Stability of Retaining Wall of Circular Shaped Portion

A small part of retaining wall of circular portion has already failed due to erosion action, but the some maintenance works were done by replacing the damaged wall portion by rock wall without making remedial works to resist the erosion action .No analysis was made for vertical walls of this portion but it is obvious unsafe due to the almost vertical slope adopted in the design.

4.5.3 Foundation Stability

The stability of embankment foundation was checked using results obtained from C.U triaxial tests, consolidation and CPT tests, and summarized in Ttable 4.6

Table 4.6 Results of CU, CPT and Consolidation Tests performed on foundation

Layer thickness (average) (m)	Consolidation results		**Friction angle** ϕ' (deg.)	Cohesion C' (kN/m^2)	Rang q_c value CPT test (kN/m^2)
	Void ratio e_o	Compression index C_c			
Clay layer 5.5	0.895	0.7223	31.75	9	2000- 7000 average 3717
Sand layer 3	-	-	-	-	6000 -14000 average (8000)

From the analysis of results of these laboratory and field tests that:

a) The allowable bearing capacity of the clay layer was found about 191 kN/m^2 by using general bearing capacity equation and about 413 kn/m^2 when using approximate method sand CPT data only.

Allowable bearing capacity of sand layer was found equal (200- 267) kN/m^2 based on an average CPT q_c value of 8000 kN/m^2.

Form above results it may be concluded that the foundation is stable from bearing capacity consideration compared with the intensity of stresses imposed by the embankment weight (about 132 KN/m^2 as maximum value).

b) The approximate method described in Appendix A was used to estimate the factor of safety against horizontal shear of clay layer. Using the data listed above the factor of safety obtained was equal to 6.7 whereas the required value is 1.5 then the foundation is safe against horizontal shear.

c) Excessive foundation settlements may lead to decreasing in the embankment level and thus increasing the probability of overtopping. The differential settlement may lead to occurrence of cracks (longitudinal and transversal) and that increasing the probability of piping failure.

Calculation were carried out to obtain the foundation settlement using results of CPT and consolidation tests, and the calculation indicated that the total settlement (elastic and consolidation settlement) was about 312 mm. The actual value of settlement may exceed the calculated values because the calculations were based on average values of embankment and foundation soil properties. Comparing the calculated settlement with the height constructed free board (2.19m), it was found that the embankment in on the safe side with respect to overtopping problem. Moreover, with the exception of the surface cracks due to erosion, no cracks appeared in the embankment body, as an indication of the differential settlement occurrence, as suggested by the results of survey works.

4.6 Seepage Characteristics

The design specified the use of a horizontal drain filler connected with toe trench to prevent and control water seepage through the embankment and it's foundation.

The inspection and field excavation did not show any evidence of the existence of any filter or drain in the embankment.

Also from the assessment and evaluation of the present condition of the embankment one can note the following points which relate to seepage through the embankment:

- There are no signs of seepage flow through the embankment. This may be related to low permeability of embankment soil and the short flood duration with low water level in the river.
- The occurrence of piping is expected in the ends of the embankment where improper compacted fill is placed in contact with the abutment walls of the two river bridge. The material used in the embankment construction was a high plasticity soil and such material would not be favorable in controlling seepage because it has high tendency to cracks when it dries after wetting.
- Calculation was carried out to determine the expected discharge (q) through embankment body and was found to be 1.68X 10^{-10} m^3 per meter length, this may be considered as a little quantity of seepage.

  The quantity seepage water passing through the foundation may be more significant than the quantity passing through the embankment. Moreover the under-seepage is harmful to the stability of downstream slope and it increases the uplift pressure specially in case of absence of horizontal drain filters.

4.7 Erosion of Embankment Slopes

4.7.1 Upstream Slope

The upstream slope must be protected against wave and current action. For this purpose the designer specified a 0.5m thick pitching layer placed on supporting gravel layer range of thickness between 0.15 -0.25m.

The existent pitching was constructed using a rock fragment (rip- rap) layer of 0.6m thickness built with cement – sand mortar and placed on supporting gravel layer about 0.2m thickness.

The constructed rip–rap layer suffered from some longitudinal and transversal cracks which could affect upstream embankment slope in form erosion and deterioration. Plate 9 and Plate 10 Appendix I show the cracks developed in the protection (rip- rap) of upstream face of embankment.

4.7.2 Protection of Downstream Embankment Slope

The downstream slope must be protected against water runoff, and for this purpose the designer specified protection layer consisting of 0.4m rock layer placed on supporting gravel layer (ranged between 0.15- 0.25 m).

Unfortunately, there was no protection constructed in downstream and it was left exposed to erosion. The erosion action is clear in forms of deep gullies and cracks due to heavy and concentrated water runoff and this feature are very clear in Plates 11 and 12 Appendix I.

A remedial action must be taken as suggested for the slope protection methods described in the Chapter 2. The embankment material is consisting of high percentage of silty and sandy soil and such soil would be moved, and so erosion may become detrimental to the embankment if such measure is not taken..

4.7.3 Embankment Crest

The designer assumed the coarse layer to be located on the top (crest) as sub- base for light traffic, the whole portion of this layer was washed and parts of it remained in some locations, saving the crest from erosion.

Generally the crest is in acceptable condition and was not significantly affected by erosion, and also it needs a suitable protection measure.

4.8 Safety Against Earthquake and Liquefaction Potential

The adverse effects of earthquake are largely dependent on seismicity of the area in which the embankment is sited, local foundation condition and type and size of the embankment.

The stability of the embankment was not checked from seismic concepts because the embankment was constructed in very low earthquake activity area (zone 2A), and located on relative dense foundation.

Moreover the embankment was built by a cohesive and unliquefiable soil. This is clear by comparing the liquefaction indication properties (percentage finer than 0.005 mm, liquid limit and water content) of constructed fill material with the corresponding values which are considered a limit value of liquefiable soil as shown in Table 4.7.

The Empirical Method (described in appendix A) prepared by Seed and developed by many researcher effort, was used to evaluate the liquefaction potential for foundation soil, using data obtained from CPT test, grain size distribution test, and seismic data of zone 2A. The set of data used are summarized in Table 4.8.

The liquefaction potential was checked at three depth (2.3m, 5.5 and 8.5m) measured from embankment base. The factor of safety against liquefaction must not less than 1, where as the calculated values are (47.4, 40 and 18.1) (Table 4.8), this values give the conclusion that the foundation soil is safe against liquefaction potential.

Table 4.7 Comparison between fill material properties and liquefiable soil properties

Limit of liquefiable material	**Actual properties of fill material**	**Comment**
Percentage of finer than 0.005≤15%	Percentage of finer than 0.005 range between 18-33% (from gain size distribution tests) with average value 26%	Safe
Liquid limit ≤ 35%	Liquid limit ranged between 43-61% (from Atterberg limit test)	Safe
Water content ≥ 0.9 (liquid limit)	- Water content of U/S portion ≈23.3% D/S portion ≈ 9.8% Crest portion ≈ 10% (from water content tests) - limit value = 0.9 (48%) = 43 %	Safe
Liquidity index ≤ 0.75	- by checking the U/S water content the liquidity index = 5.1%	Safe

Table 4.8 Summary of data and results of liquefaction potential evaluation

Checked depth (m)	**N_{spt} value (from CPT test)**	**Percentage of finer**	**Seismic data** (for the embankment area)	**FS_{liqu} in 7.5 M_{zones}**	**Transefer Fs_{liqu}. to 5.5 M $_{zone}$**
2.3	8	77.8%	Zone 2A:peak ground acceleration =0.12g and assumed earthquake magnitude =5.5	17.2	47.4
5.5	5	89.7%		14.8	40
8.5	5	11%		6.6	18.1

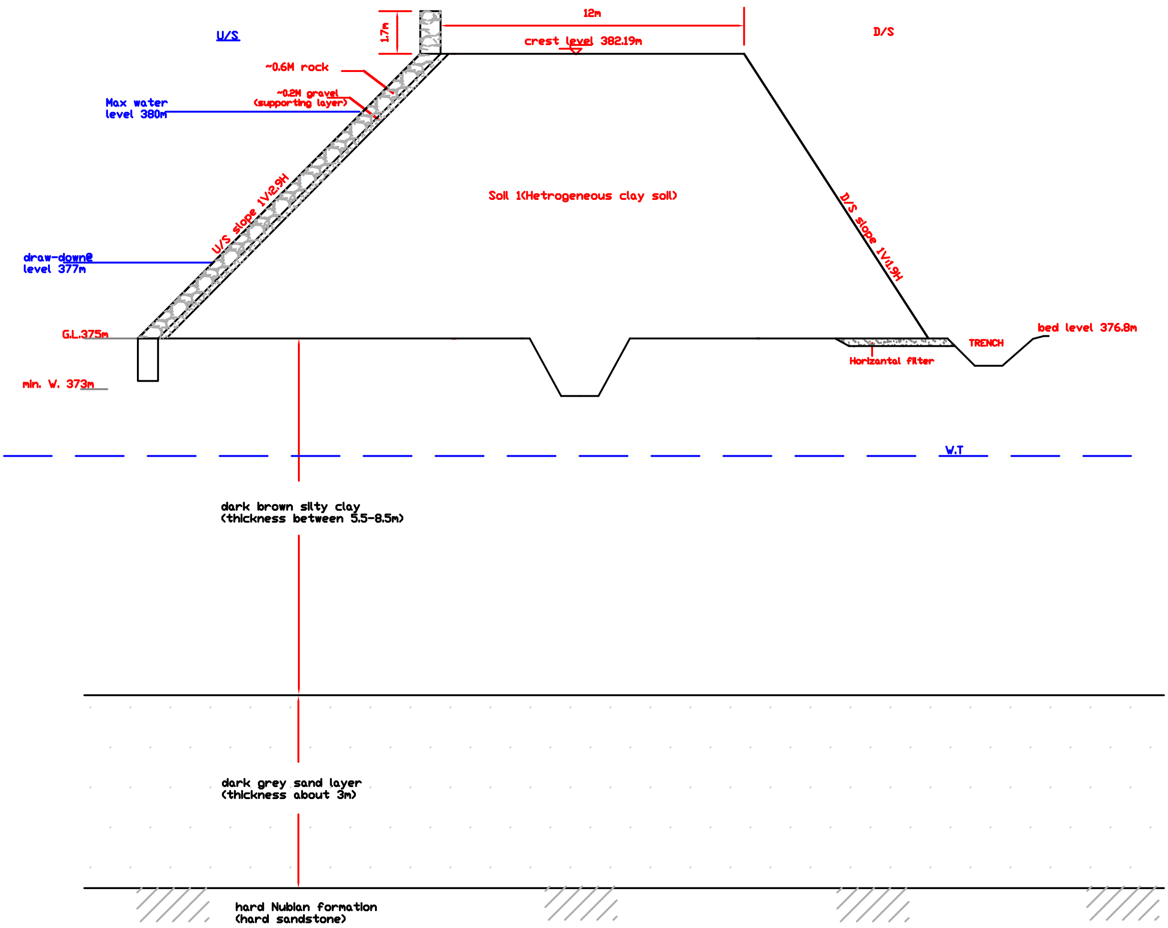
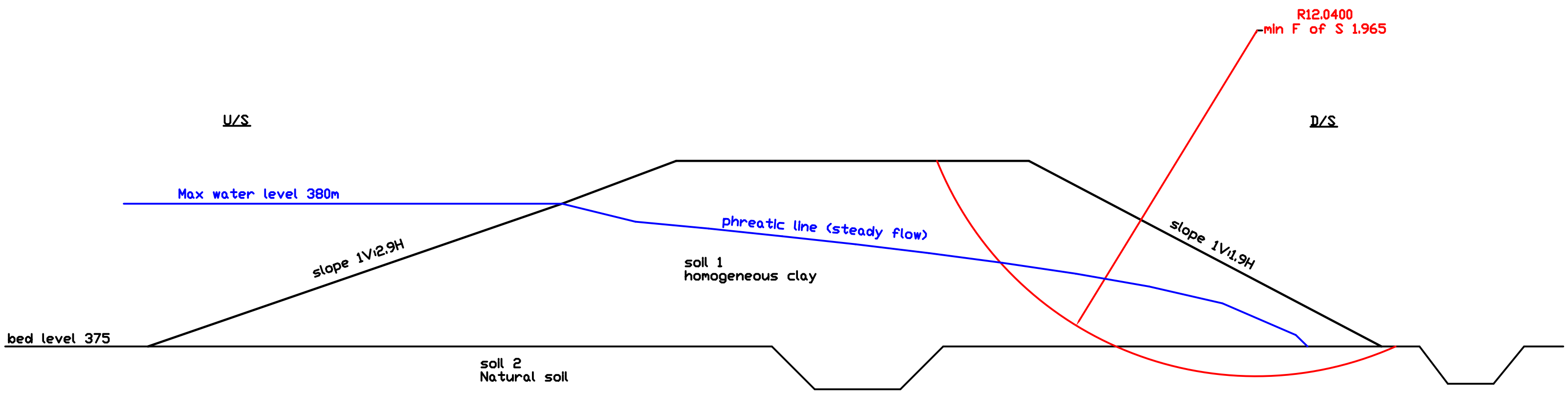


Figure 4.1: Existing section (dimention and feature) of Embankment and foundation



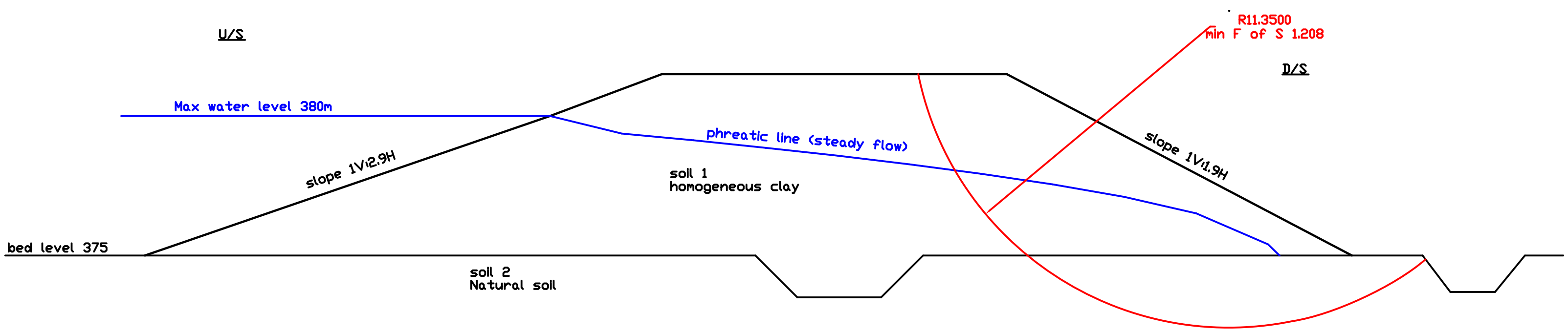
D/S slope stability (steady flow)

average value:

soil 1		
cohesion(Kn/m2)	friction	Unit Wt.(Kn/m3)
17.6	23	18.3

soil 2		
cohesion(Kn/m2)	friction	Unit Wt.(Kn/m3)
9	31.75	19

Figure 4.2 a: Critical D/S sliding surface for steady state flow (average soil properties)



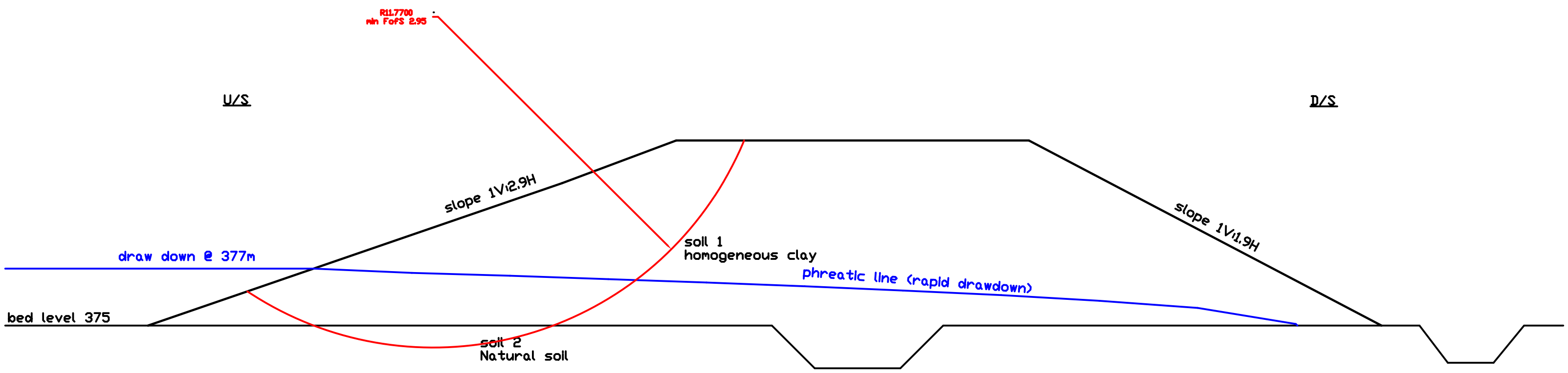
D/S slope stability (steady flow)

min value:

soil 1		
cohesion(Kn/m2)	friction	Unit Wt.(Kn/m3)
10	21	16.2

soil 2		
cohesion(Kn/m2)	friction	Unit Wt.(Kn/m3)
0	25	18

Figure 4.2 b: Critical D/S slope sliding surface for steady flow (min soil properties)



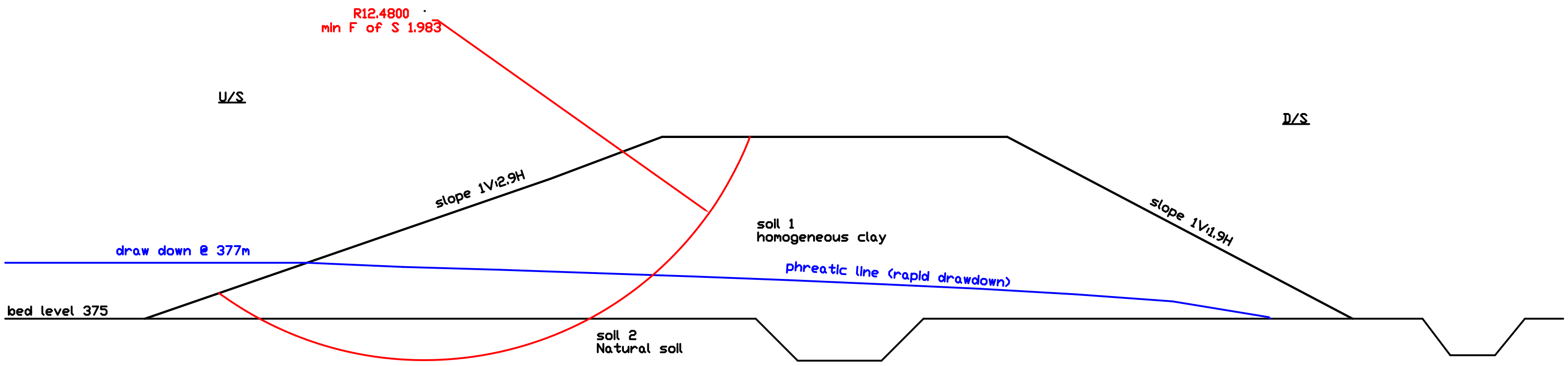
D/S & U/S slope stability (rapid drawdown)

average value

soil 1		
cohesion(Kn/m2)	friction	Unit Wt.(Kn/m3)
17.6	23	18.3

soil 2		
cohesion(Kn/m2)	friction	Unit Wt.(Kn/m3)
9	31.75	19

Figure 4.3 a: Critical U/S sliding surface for drawdown state flow (average soil properties)



U/S slope stability (rapid drawdown)

min value:-

soil 1		
cohesion	friction	density
10	21	16.2

soil 2		
cohesion	friction	density
0	25	18

Figure 4.3 b: Critical U/S sliding surface for drawdown state flow (min soil properties)

CHAPTER 5

CONCLUSIONS AND RECOMMENDATION FOR FUTURE STUDIES

5.1 Conclusions

Chapter 4 reviewed the current condition of the specific embankment under consideration and presented an analysis of the data and information which represent the features and component of the existing embankment and its foundation.

The following paragraphs summarize the main conclusions drawn from analysis of the evaluation and technical assessment study of the existing embankment

i. There were notable differences between the executed embankment section from the proposed geometry specified by the designer. The designed and exacted embankment section differed in

- The constructed embankment length is longer than is given in the drawings
- The constructed freeboard increased by 46% from design value, due to increment in constructed embankment height by 0.7m from proposed value
- The actual embankment crest equal 12m and its overestimated.
- The downstream and upstream slopes are slightly steeper and flatter respectively.
- There were circular shape and wave wall supplemented to embankment but not mention in design.

ii. No adequate field investigations were conducted at the embankment site and borrow area prior to design and construction of the embankment, and the construction was accompanied with lack of professional supervision staff .This was clear in the unstable materials used in construction which were heterogeneous and not meeting the designer specification. The material used for constructing the embankment was a mix of clay and silt clay soils has undesirable properties like high plasticity, weak to resist erosion but it extremely impervious and non-dispersive. The lack of quality control during construction stage was clear in improper compaction therefore just 44% of field compaction tests performed on embankment reaches the recommended compaction value (over 95%).

iii. The stability analysis of upstream and downstream slopes carried out in this study showed that the upstream slope was safe during steady seepage and rapid drawdown condition, and the factor of safety exceeds the values recommended by US corps of engineers. The downstream slope was safe against sliding during rapid drawdown and steady seepage conditions, but the analysis shows the downstream is not safe in steady seepage case when applying the minimum value of shear parameter. No analyses was made for the vertical walls of downstream and upstream slopes of the circular shape portion but were considered unsafe.

iv. The foundation soils were two thick soil layers located beneath the embankment, one is high plasticity clay and the second is very loose silty sand. The foundation was considered stable from bearing capacity viewpoint when compared with stress imposed by the embankment weight. The total settlement of foundation was found to be 312mm most of which might have already taken place. There are no cracks which give a sign about differential settlement in the embankment body.

v. There were no effective seepage systems installed in the existing embankment except the toe trench where the designer specified a horizontal filter connected with toe trench .There were no signs of seepage flow through the embankment, but generally the seepage system was not satisfactory.

vi. The designer proposed rock pitching to resist erosion in upstream and downstream embankment faces, but the pitching was performed in the upstream face only. The downstream was left exposed to erosion and as a result of this the downstream slope suffered from occurrence of deep gullies and cracks which could affect the stability of downstream slope, and also the upstream protection suffering from cracks. The embankment crest was generally acceptable and it surface was not significantly affected by erosion.

vii. The embankment fill material is not susceptible to liquefaction and no risk of liquefaction potential .The potential of liquefaction was also checked for the embankment foundation soil and was found safe.

viii. From above conclusions one can reach to fact that the existing embankment is not suitable and capable to protect the costly urban area in landside planned to be constructed in the near future. Remedial actions must be done to reduce the hazards related to major excessive seepage or possible hydraulic and structural failure. These should cover the following:

- Repair cracks on upstream face of embankment.
- Protect the downstream embankment slope by suitable method and flatten the slope.
- Treat the circular shape portion by flatting or berms the slopes, and make an effective surface drainage system.
- Place new compacted layer on the embankment crest.
- Always keep the longitudinal downstream trench clean to drain all collected water in downstream side.

ix. These above remedial measures must be considered as temporary and planning should be started to design and construct a new structure, correctly and appropriately to protect the urban area.

5.2 Recommendations for Future Studies

The future studies should go deep in the following fields:

- The quantity and affects off under-seepage in stability of embankment.
- The phased programs to carry the quality control, monitoring and surveillance during construction and operation of the embankments.
- The use of alternative structures like steel piles to safe the urban area from floods and the cost of each option.

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APPENDIX (A)
ASSISTANT THEOREMS

Appendix A
Assistant Theorems

A.1 Stability Analysis

There are many methods which can be used in the analysis of stability of slopes of embankment dams, but in this appendix just the Swedish circle method covered.

A.1.1 Swedish Circle Method of Slope Stability Analysis

In this method the potential failure surface is assumed to be cylindrical, and the factor of safely against sliding is defined as the ratio of the average shearing strength, as determined by coulomb's equation $S = c + s\tan\phi$ to the average shearing stress determined by static on the potential sliding surface.

In order to test the stability of the slope, a trial slip circle is drawn, and the soil material above assumed slip surface is divided into convenient number of vertical strips or slices. The weight (W) of each assumed to act at its centre. If this weight of each slice is normal (N) and tangential (T) components, then the normal component will pass through the centre of rotation (q) , and hence does not cause any driving moment on the slice. However, the tangential component (T) cause a driving moment = (T* r), where (r) is the radius of the slip circle. The tangential components of the far slices at the base may cause resisting moment; in that case T is considered negative.

If C is the unit cohesion and (△ L) is the curved length of each slice, the N the resisting force, from coulomb's equation is $C.\Delta L + N\tan\phi$ (Figurer A.1)

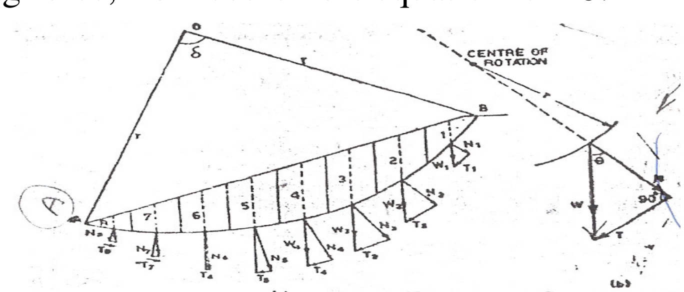

Figure A.1 Slip circle method

For entire slip surface AB, we have:

Driving moment $Md = r.\sum T$

Resisting moment $Mr = r.[C.\Delta L + \tan\phi \sum N]$

$\sum T$ ≡ sum at all tangential components.

$\sum N$ ≡ sum at all normal components.

$\sum \Delta L = \frac{2\pi r \delta}{360^o}$ ≡ length AB of slip circle = L

hence factor of safety against sliding is

$$F = \frac{Mr}{Md} = \frac{C.L + \tan\phi \sum N}{\sum T} \qquad \text{(I)}$$

A.1.2 Method of Locating Centre of Critical Slip Circle

In order to reduce the number of trials to find the centre of critical slip circle, Fellenious has given a method of locating the locus on which the probable centre may lie.

From Figurer A.2 and for homogenous $C-\phi$ soil, centre of critical slip circle lies on a line PQ, in which the point Q has its coordinates H downwards from toe and 4.5 H horizontally away, the other point P is obtained with the help of directional angles ά and β given in table below

Slope angle (i)	Directional angles	
	ά	β
60	29	40
45	28	37
33.8	26	35
26.6	25	35
18.4	25	35
11.3	25	35

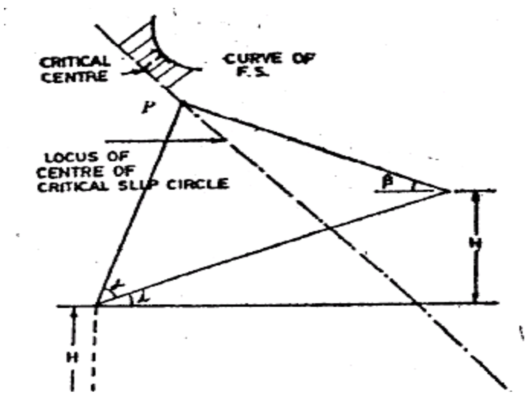

Figure A.2 Method of locating center of critical slip surface

When the line PQ is obtained, trial centres are selected and factor of safety corresponding to each centre is calculated from equation (I).

These various factors of safety so obtained are plotted as ordinates (Figurer A.2) on the corresponding centres, and a smooth curve is obtained. The centre corresponding to the lower factor of safety is then the critical center.

A.1.3 Stability of Downstream Slope during Steady Seepage:

The factor of safety in this case is given by

$$F.S = \frac{CL + \tan\phi \sum (N - U)}{\sum T} \qquad \text{(II)}$$

When $\sum U$ is the total pore pressure on the slip surface.

Figurer A.3 shows the downstream slope of earth for provided with a horizontal filter at its toe

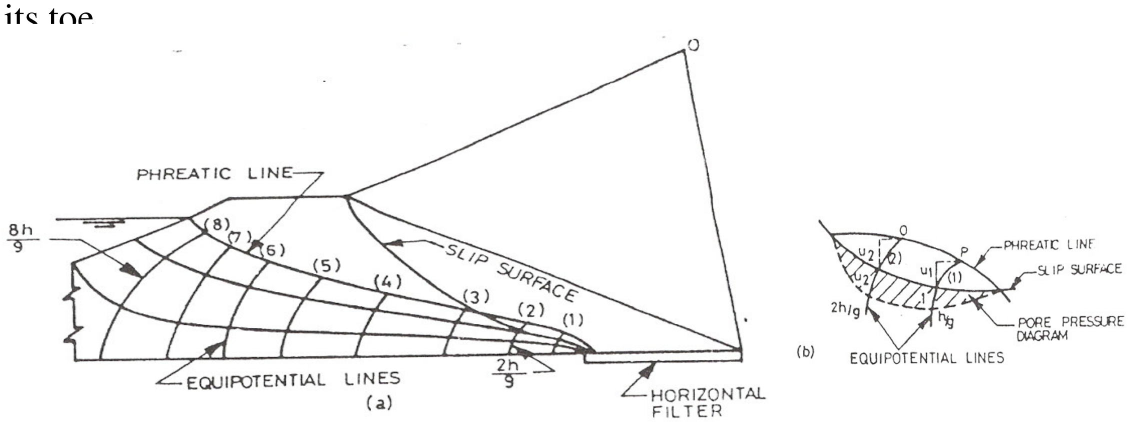

Figure A.3 Steady seepage condition

The boundary pore pressures acting on the slip surface are obtained from the flow net Figure A.3b shows the enlarged view, the pore pressure (u_1) at point-1 where the equipotential line cuts the slip surface is equal to the vertical distance between the point-1 and the point P where the equipotential line intersects to phreatic line. As the pore pressure acts normal to the surface, a line equal to u_1 is drawn normal to the slip surface at point -1. Likewise, the pore pressure u_2 at point-2 is found. The pore pressure diagram is drawn joining the extremities of all these lines. The pore pressure diagram is shown hatched in Figure A3b, however; in this case the total weight of the slice is due to bulk unit weight above the phreatic line and the saturated unit weight below the phreartic line.

Equation (II) can be written as:

$$F.S = \frac{\sum C\Delta L + \sum (N - uL)\tan\phi}{\sum T}$$

Where (u) is the average pore pressure on the slice and L is the length of the base of slice $(L = \Delta L = b\sec\propto)$.

A.1.4 Stability of Upstream Slope during Sudden Drawdown

The critical condition for the stability of the upstream slope of an earth dam is when there is a sudden drawdown in the reservoir upstream. If soil is of low permeability, no appreciable change in the saturation level inside the slope takes place when the reservoir level goes down. The weight of water which is still present in the soil tends to cause sliding of the wedge, as the water pressure which was acting on the upstream slope to balance this weight has been suddenly removed. According to another interpretation, the shearing resistance of the soil is considerably reduced due to pore pressure existing in the soil, whereas the disturbing force due to saturated weight of the soil remains the same.

In order to calculate the factor of safety the equation below can be used

$$Fs = \frac{CLa + \tan\phi \sum N'}{\sum T}$$

In which the N' component are computed with respect to the submerged density γ' of the upstream slope, while T component are calculated with respect to the saturated density.

A.1.5 Analysis for Sloughing of Upstream Slope during sudden Drawdown

These is another approximate method for finding the factor of safety against sliding or sloughing of upstream slope during sudden drawdown, refer to Figure A.4

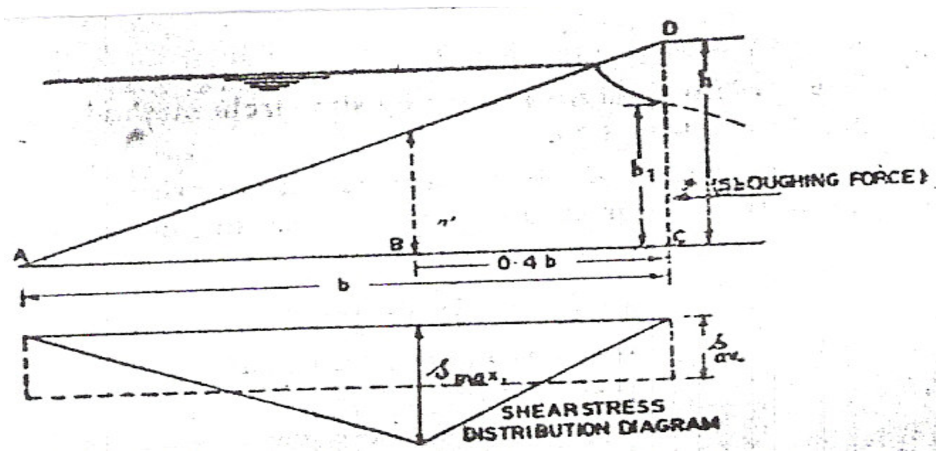

Figure A.4 Sloughing of u/s slope during sudden drawdown

Let $\phi' \equiv$ Submerged unit weight of the material in U/S portion of dam.

γ_s $\equiv$ Saturated unit weight of the material in U/S portion of dam

b $\equiv$ Horizontal length of the slope AD

P $\equiv$ Total horizontal shear or sloughing force on the U/S portion of dam

S_{av} $\equiv$ Average unit shear over upstream of dam

$S_{\max}$ $\equiv$ Maximum unit shear over upstream of dam

h, h_1 and h heights as marked in Figure A.4

Then the horizontal shear force is given by:

$$P = \frac{\gamma_s h^2}{2} \tan^2\left(45 - \frac{\phi}{2}\right) + \frac{\gamma_w h_1^2}{2}$$

$$Sav = p/b$$

$$S_{\max} = 2Sav = \frac{2p}{b}$$

The maximum shear occurs at distance of 0.4 b from the U/S point, as shown

Let $R =$ shearing resistance = $N \tan\phi + cb$

Where $N = \gamma' A$

A = area of U/S slope = ½ bh

Factor of safety = $\frac{R}{P}$ "this should be greater than 2".

A.1.6 Stability of Foundation against Shear

Foundation consisting of fine, loose, cohesionless materials or of unconsolidated clays and silts may be very weak in shear and may require through investigation. The present method of finding factor of safety of foundation against horizontal shear is an approximate one, and based on the assumption that earthen material has an equivalent liquid unit weight which would produce the same shear stress as the material itself.

From Figure A.5

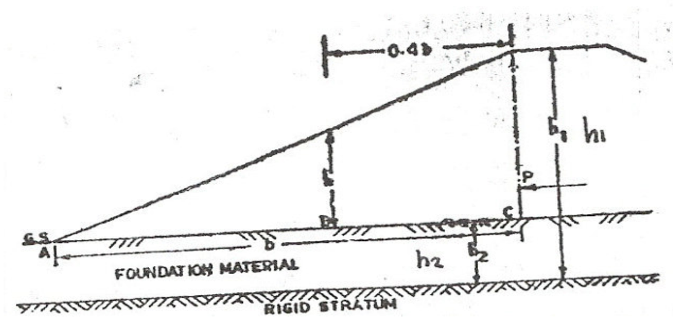

Figure A.5 Foundation shear

IF P ≡ total horizontal shear down to rigid boundary, then

$$P = \frac{h_1^2 - h_2^2}{2} \gamma_m \tan^2\left(45 - \frac{\phi_1}{2}\right)$$

Where $\gamma_m \tan^2\left(45 - \frac{\phi_1}{2}\right) \equiv$ Equivalent liquid unit weight

$\phi_1 \equiv$ Equivalent angle friction given by

$$\tan\phi_1 = \frac{\gamma_m h_1 \tan\phi + c}{\gamma_m h_2}$$

Where ϕ and c are shear parameters for the foundation material.

γ_m = mean unit weight of the dam and foundation weighted in proportion to the depth of each.

$$\gamma_m = \frac{\gamma_D(h_1 - h_2) + \gamma_F h_2}{h_2}$$

Where

$\gamma_D \equiv$ Unit weight of dam material

$\gamma_F \equiv$ Unit weight of foundation material

Now average unit shear = $S_{av} = {}^{p}\!/_{b}$

Maximum unit shear = $S_{\max} = 1.4 S_{av}$ "occurs at 0.4 b from c"

Let $S_1 =$ Unit shear strength below toe (at A)

$$= C + \gamma_m h_2 \tan\phi$$

$S_2 =$ Unit shear strength at point c

$$= C + \gamma_m h_2 \tan\phi$$

Average shear strength $S = \dfrac{S_1 + S_2}{2}$

Hence overall factor of safety against shear $Fs = \dfrac{S}{S_{av}}$ "should be greater than 1.5".

A.2 Estimation of Freeboard

For detailed design of freeboard the following summarized the safe freeboard calculation.

i. Fetch

Fetch is the distance (km) between the dam and land surrounding the body of water, the calculation of fetch depends on reservoir's topography.

ii. Design Wind

Design wind estimates should be obtained from the bureau of meteorology or equivalent organization. Correction must be to wind velocity over land to use the velocity of wind over water, correction made by using table below:

Wind relationships-water to land (USBR 1981)

Effective fetch(km)	0.8	1	1.6	3.2	4.8	6.4	3 (or more)
Wind velocity ratio $\left(\frac{overwater}{overland}\right)$	1.08	1.0925	1.13	1.21	1.26	1.28	1.3

iii. Wave Height

For the estimation of minimum freeboard the significant wave height in meters (Hs), which is can be estimated from Figure A.6

For the normal freeboard computation, the run up should be calculated using the

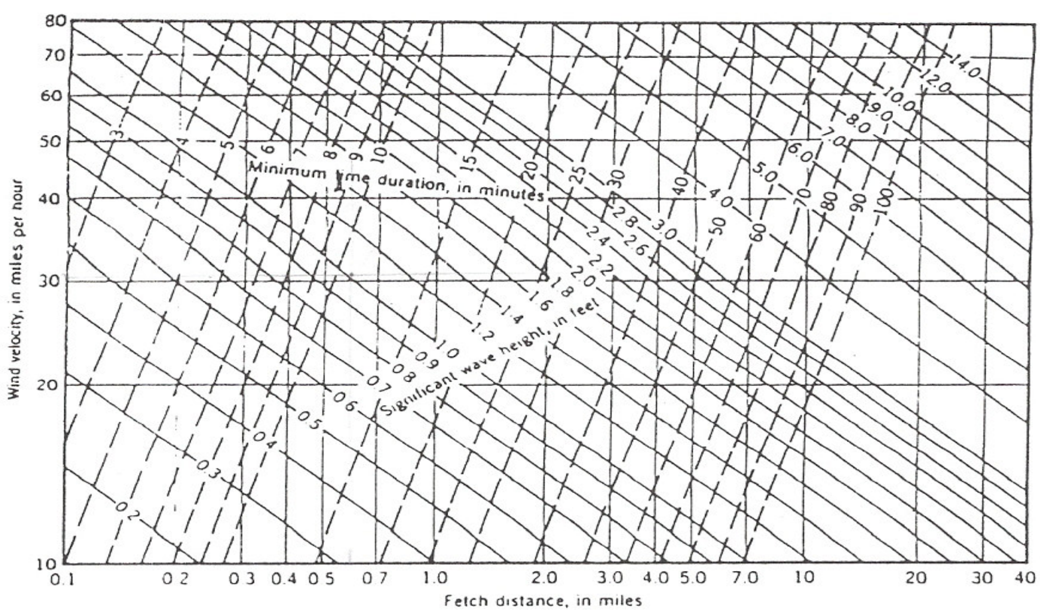

Figure (A-6) Wave heights and minimum duration wind (Saville et al. 1962 and USBR 1981). Note 1 mile per

A.3 Earthquake and Liquefaction Analysis

A.3.1 Effect of Earthquake on Embankment Dams

Earthquakes impose additional loads on to embankment dams over those experiences under static conditions. The earthquake loading is of short duration, cyclic and involves motion in the horizontal and vertical direction. Earthquake can affect embankment dams by causing any of the following:

1. Settlement and cracking of embankment.
2. Reduction of freeboard due to settlement which may overtopping.
3. Instability of slopes.
4. Differential movement between the embankment, abutment and spillway structure, increasing the likelihood of leakage and piping failure.
5. Liquefaction or loss of shear strength in the embankment and it's foundation due to increase in pore pressure.
6. Overtopping of the dam be waves due to earthquake.

The potential for such problems depend on

i. The seismicity of the area in which the dam is sited.
ii. Local foundation and topographic conditions.
iii. The type of dam.
iv. The size of the embankment.

A.3.1.1 Measurement of Earthquake Strength

Earthquake is measured in terms which are mentioned in the following

i. Magnitude

This is quantitative value obtained from seismographs, and reflects the total energy radiating from the focus of an earthquake. Earthquake with magnitude of less than 3 or 4 will usually not case any felt effect, and earthquake with magnitude less than about 5 will usually not cause any damage. The maximum recorded magnitude is ≈ 8.9.

ii. Intensity

Earthquake intensity is qualitative value based on the response of people and objects to the earthquake. The intensity depends on distance from earthquake, ground conditions and topography.

iii. Acceleration

For the design of dams, the horizontal of vertical acceleration induced by the earthquake at the base of the dam is usually required. Information is best obtained from accelerograph measurements at the dam site, but in many cases will be obtained from records of sites with similar geological conditions.

The effect of an earthquake is attenuated with distance from the epicenter. There are several equations available based on recorded events, e.g. Estera and Rosenblueth suggest that ground acceleration (at the project site) is given by:

A = 2000 $e^{0.8M}.R^{-2}$

Where:

A ≡ Peak acceleration as % of acceleration due to gravity

R ≡ Focal distance in (km)

M ≡ Earthquake magnitude

A.3.2 Liquefaction of Dam Embankment and Foundation

One of the most critical issues relating to the effect of earthquake on dams is whether liquefaction of dam or the dam foundation.

Liquefaction is the phenomenon where excessive deformation or movement occurs as result of transient or repeated disturbance of cohesion less soils, this will be accompanied by an increase in pore pressure, and partial or total loss of shear strength.

Flow failures and deformation failures are forms of liquefaction, flow failure describe the condition where the soil mass deforms continuously under a shear stress equal to the static shear stress applied to it, e.g. slope instability, total bearing capacity failure. Deformation failures involve large permanent displacement or settlement, but the earth mass remains stable without great changes of geometry.

Phenomena which occur as result of liquefaction include:

- Soil boils: formed by water flowing upward to the surface from a zone of high water pressure. Soil may flow with water.
- Flow failures of the slopes of a dam, due to reduction in strength.
- Lateral spreads.
- Loss of bearing capacity.
- Ground settlement

A.3.2.1 Evaluation of liquefaction potential for foundation

Evaluation of liquefaction potential for embankment foundation is based on empirical method which sorted by Seed and co-researcher and later developed by many researchers. The steps of this evaluation are listed below and attached with flow chart in figure A.7.

i. Determine the cyclic Resistance Ratio (CRR)

Firstly, compute corrected value of $(N_1)_{60}$ from measured SPT blow counts or CPT tests.

The corrected $(N_1)_{60}$:

$(N_1)_{60} = N_{SPT} \,.\, C_N \,.\, C_E \,.\, C_B \,.\, C_S \,.\, C_R$

Where $C_N \equiv$ first correction factor $= \sqrt{\dfrac{p_a}{\delta'_{v0}}} \leq 2.0$

$p_a \equiv$ one atmosphere of pressure (101.325 MP)

$\delta'_{v0} \equiv$ vertical effective stress of the depth of N_{SPT}

$C_E \equiv$ used to correct the measured SPT below count for level of energy delivered by the SPT hummer. Using 60% of the theoretical energy.

$C_E = \frac{ER}{60} =$ (actual energy delivered to the top of drill rod) /60

$C_B \equiv$ the correction factor for bore hole diameter outside the recommended range, assumed to by equal =1

$C_S \equiv$ the sample correction – ranged between 1-1.2 (assumed =1.0)

$C_R \equiv$ loss of energy factor and depend on failing length of the rod in SPT test, and compute form :

For $z \le 3m \rightarrow C_R=0.75$

$3< z < 9m \rightarrow C_R= (15+z)/24$

For $z \ge 9m \rightarrow C_R= 1.0$

Where z is the length of the drill rod in meters

Secondly, compute the clean sand equivalent $(N_1)_{60}$ from equation :

Clean- sand equivalent $(N_1)_{60} = (N_1)_{60}+\Delta (N_1)_{60}$

Correction factor $\Delta (N_1)_{60}$ computed from linear function

- for $Fc \le 5\% \rightarrow \Delta (N_1)_{60}=0.0$
- for $5< Fc < 35\% \rightarrow \Delta (N_1)_{60} =7(Fc-5)/30$
- for $Fc > 35\% \rightarrow \Delta (N_1)_{60} = 7.0$

Where Fc is fine content (percentage finer than 0.075mm)

Thirdly, obtain the cyclic resistance ratio (CRR) based on computed $(N_1)_{60}$

This done using figure A.8 or equation

$$100.CRR_{M=7.5} = \frac{95}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{1.3} - \frac{1}{2}$$

Where $CRR_{M=7.5}$ is cyclic resistance ratio for earth quake magnitude equal 7.5

The $CRR_{M=7.5}$ must be adjusted for the magnitude of earthquake under consideration, this is done using equation

$$CRR = CRR_{M=7.5} * MSF$$

Where (MSF) is magnitude scaling factor which is obtained from:

- for M<7.0 $\rightarrow$ MSF= $10^{3} * M^{-3.46}$
- for M $\geq$ 7.0 $\rightarrow$ MSF $=10^{2.24} * M^{-2.56}$

Where M is earthquake magnitude.

ii. Determine the cyclic stress Ratio (CSR) induced by earthquake

The cyclic stress ratio computed with simplified equation suggested by Seed et and Idriss, where

CSR = $0.665 \frac{a_{\max}}{g} \frac{\delta_{vo}}{\delta`_{vo}} .rd$

Where g ≡ acceleration gravity (9.81 m/s^2)

δ_{vo} ≡ Total overburden stress at depth of interest.

$\delta`_{vo}$ ≡ Effective overburden stress at depth of interest.

$a_{\max}$ ≡ Peak acceleration induced by earthquake

rd ≡ stress reduction factor, and it depends on depth and obtained from figure A.9

iii. Compare the (CRR) with (CSR)

Compute the factor of safety against liquefaction using

$$Fs_{liq} = \frac{CRR}{CSR}$$

If $Fs_{liq} \leq 1.0$ there is liquefaction potential

$Fs_{liq} > 1.0$ indicates No liquefaction

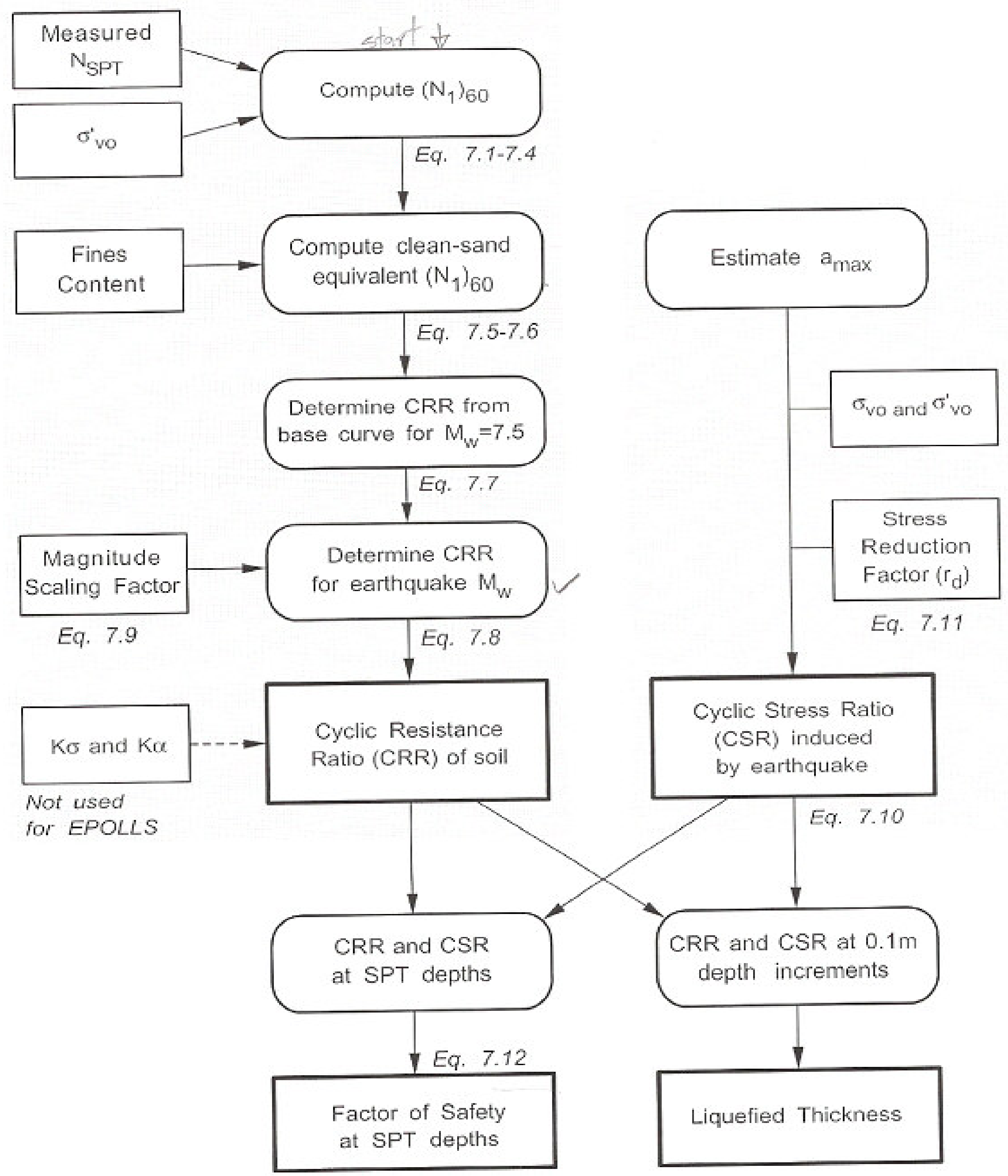

Figure A.7 flowchart for evaluation the liquefied thickness of the soil based on SPT blowcounts

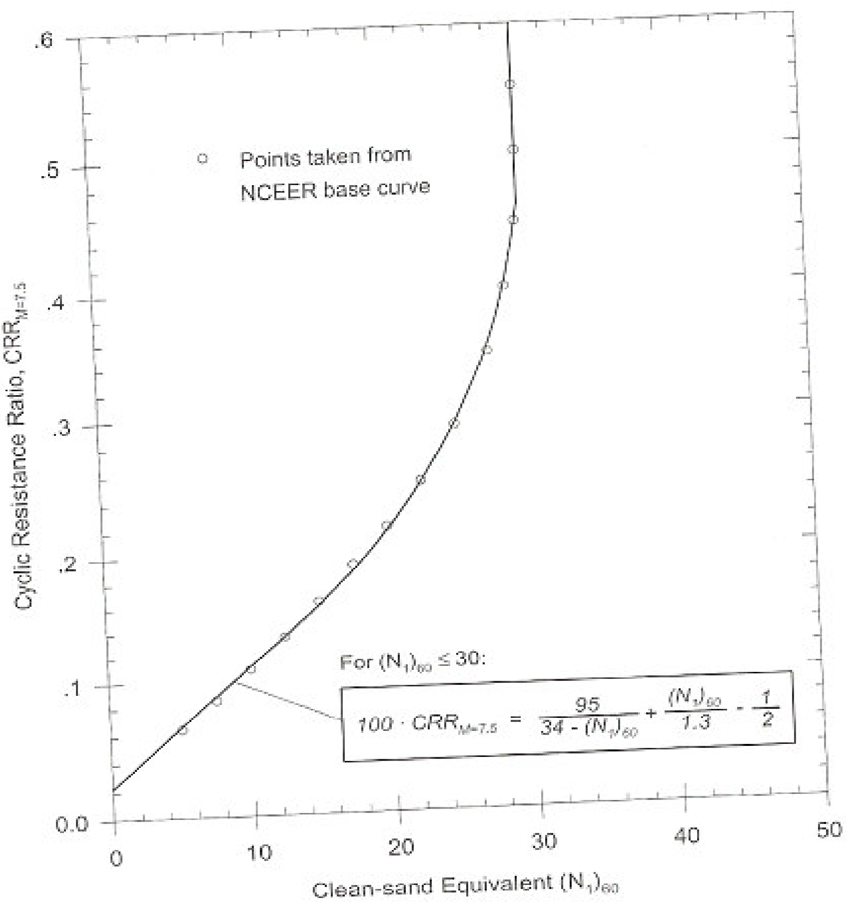

Figure A.8 Base curve for getting $CRR_{M=7.5}$ from corrected SPT blowcount.

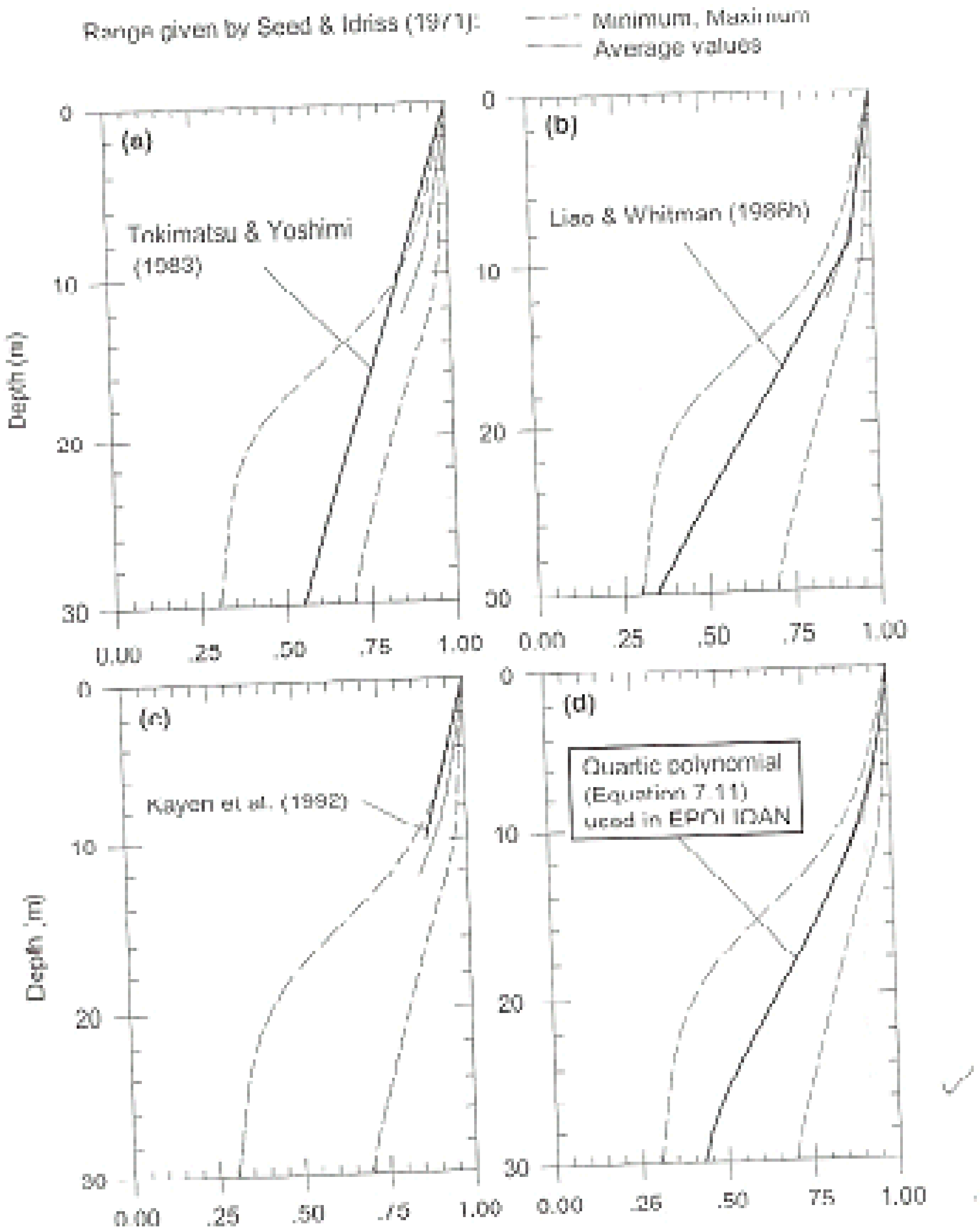

Figure A.9 Approximations for the stress reduction factor (r_d)used in computing CSR

APPENDIX (B)

BOREHOLES LOGS

APPENDIX (C)
CONE PENETRATION TEST RESULTS

Embankment

Cone Penetration Test Results

1

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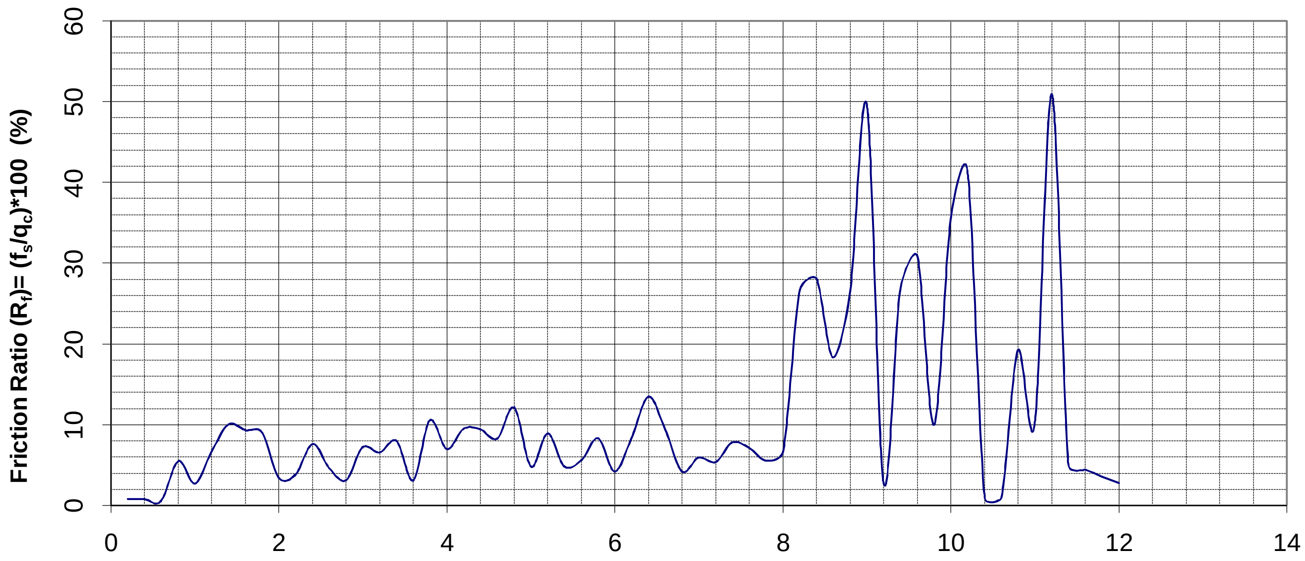
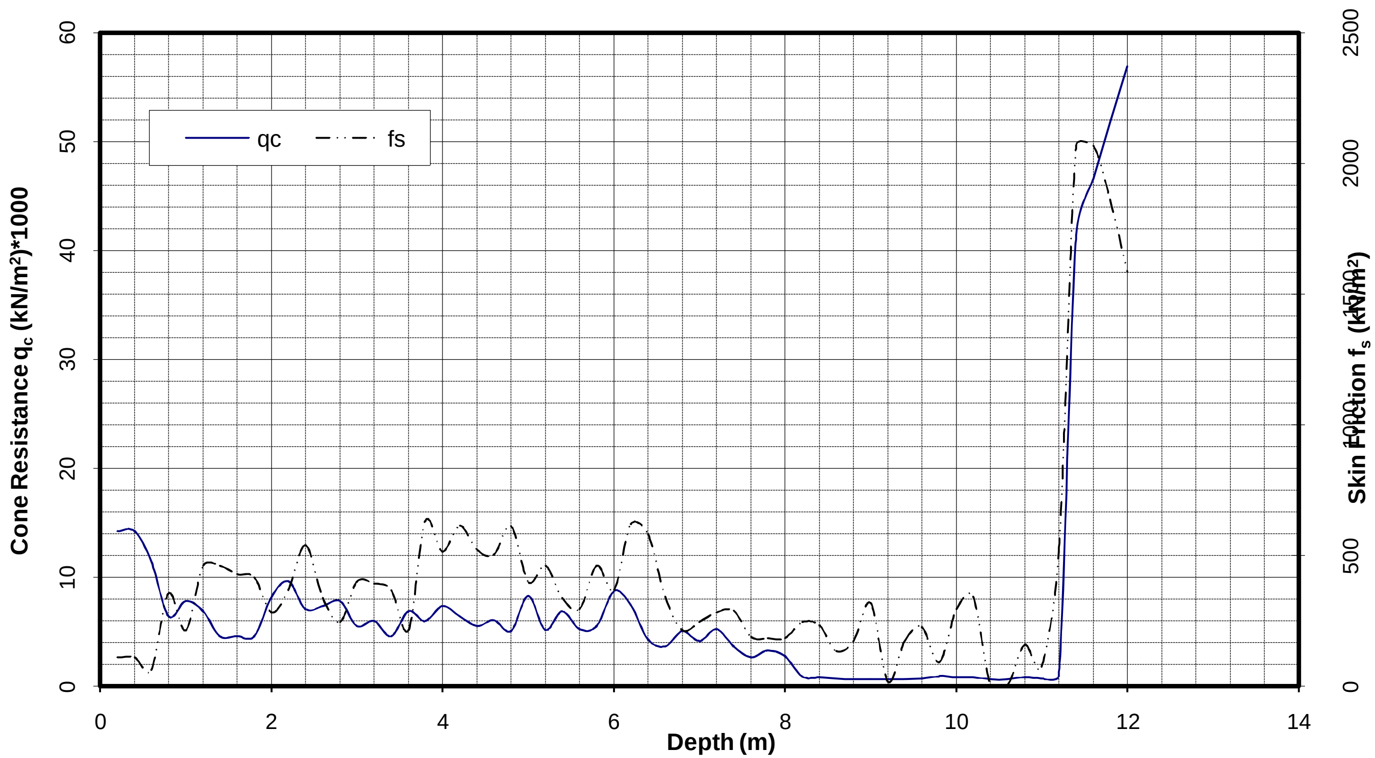
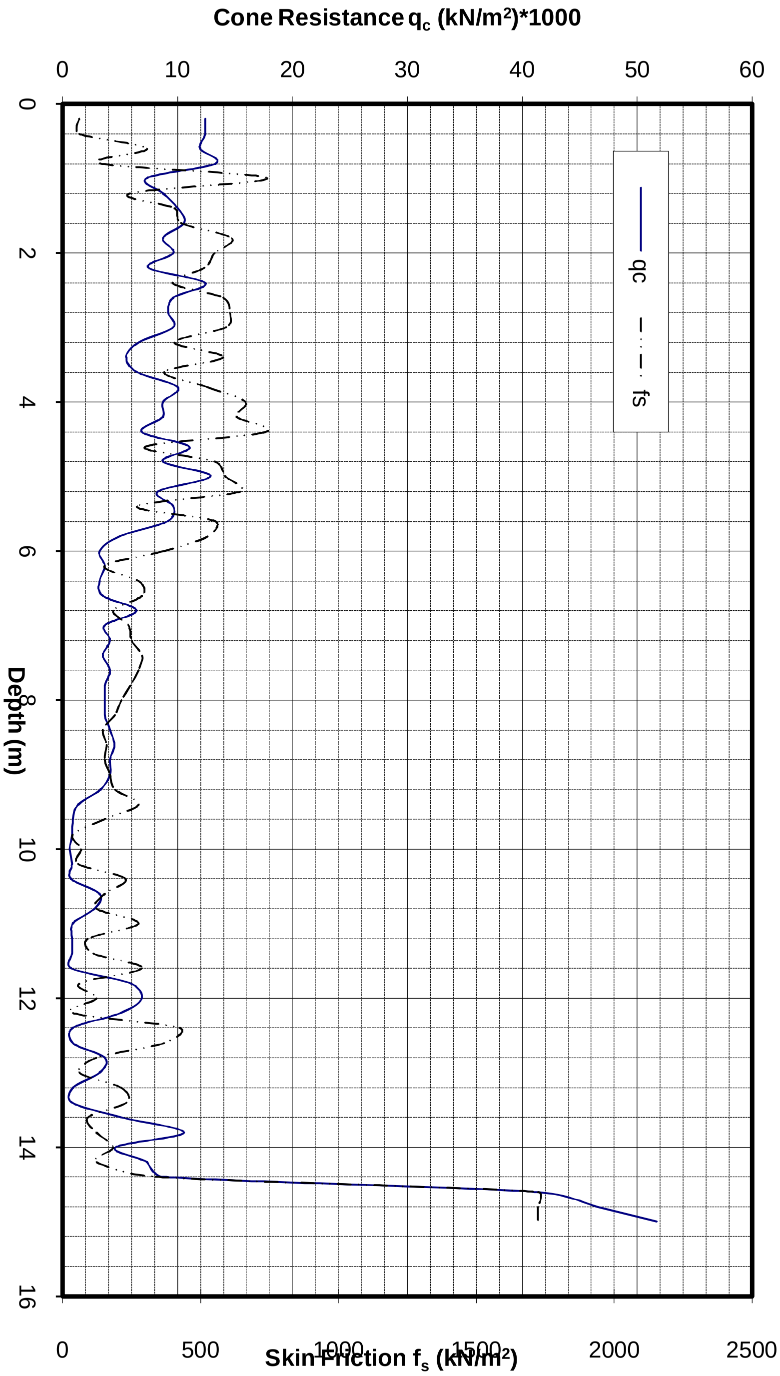
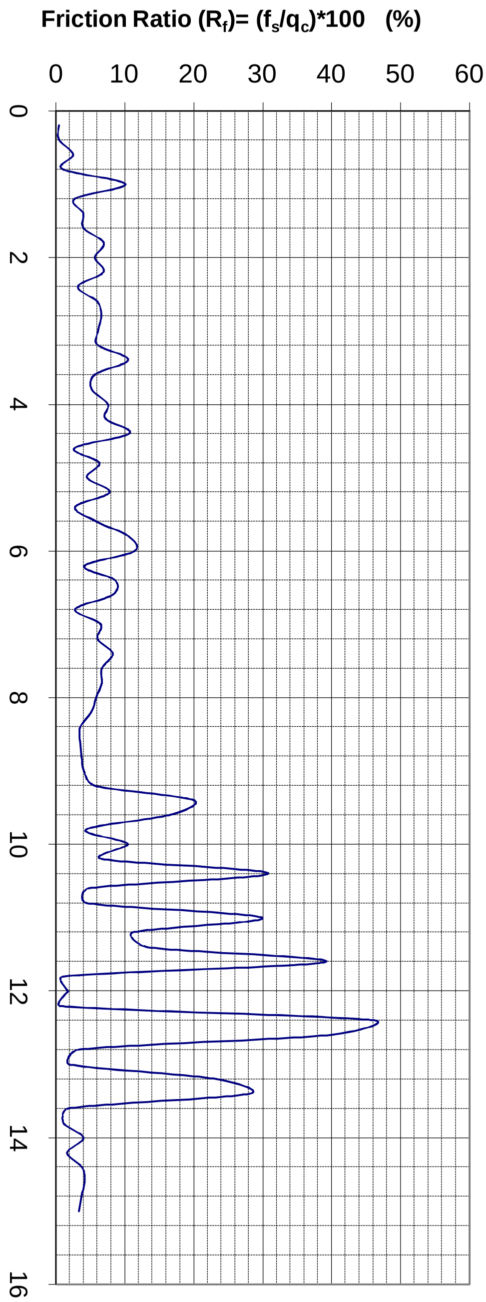




Building & Road Research Institute University of Khartoum	Cone Penetration Test Results	Embankment

2



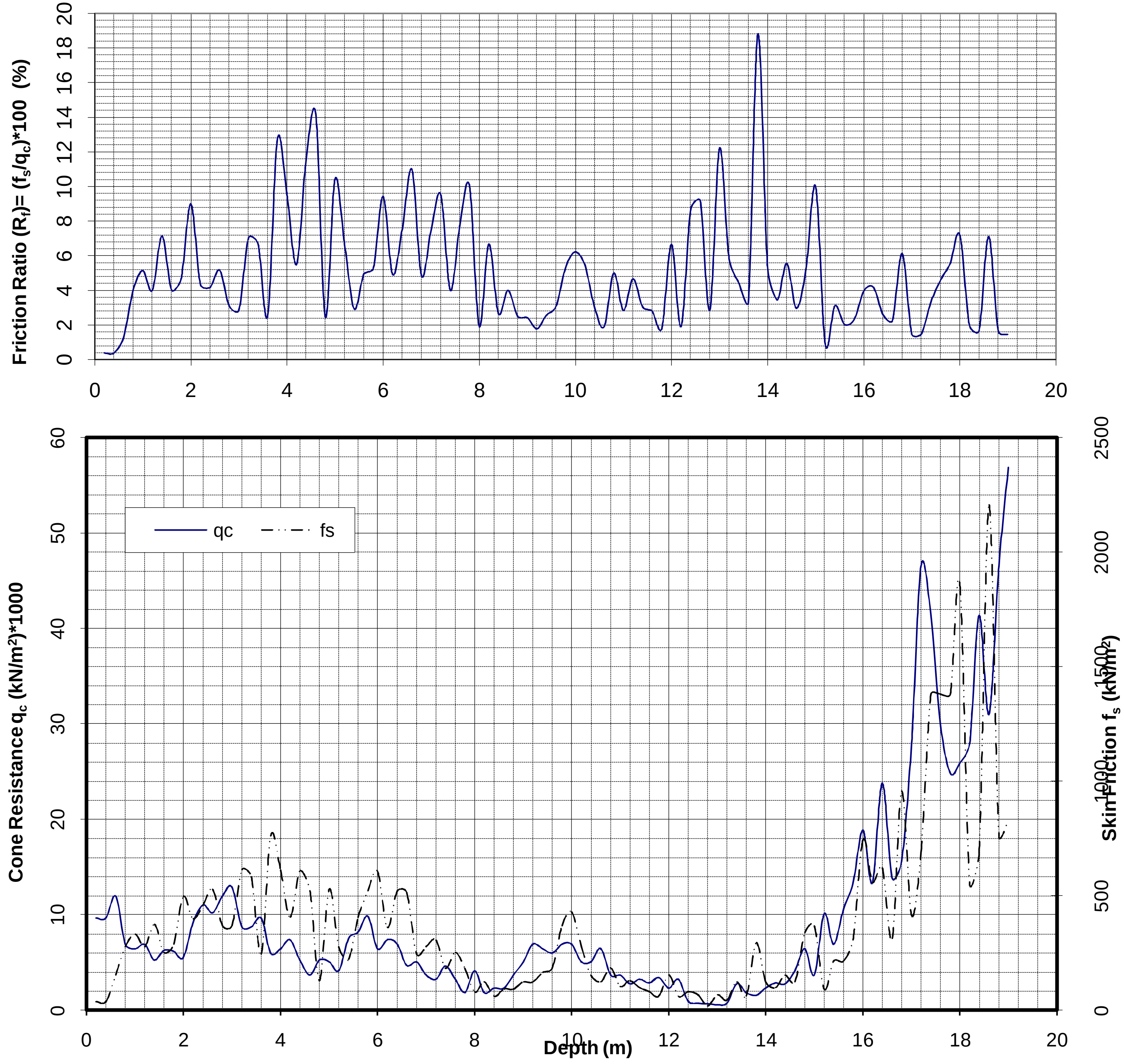


Embankment

Cone Penetration Test Results

3

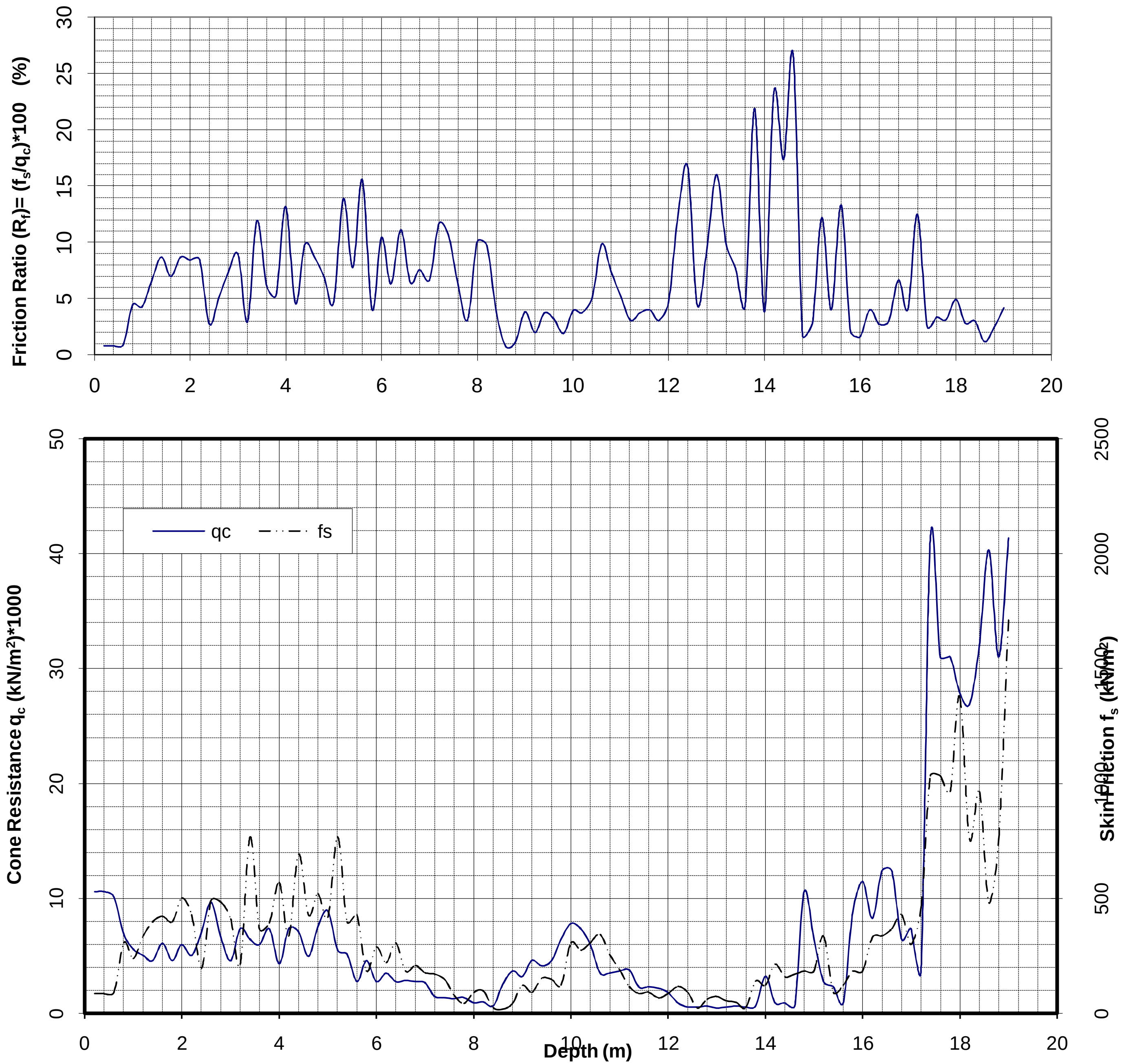
Building & Road Research Institute
University of Khartoum



Embankment

Cone Penetration Test Results

4



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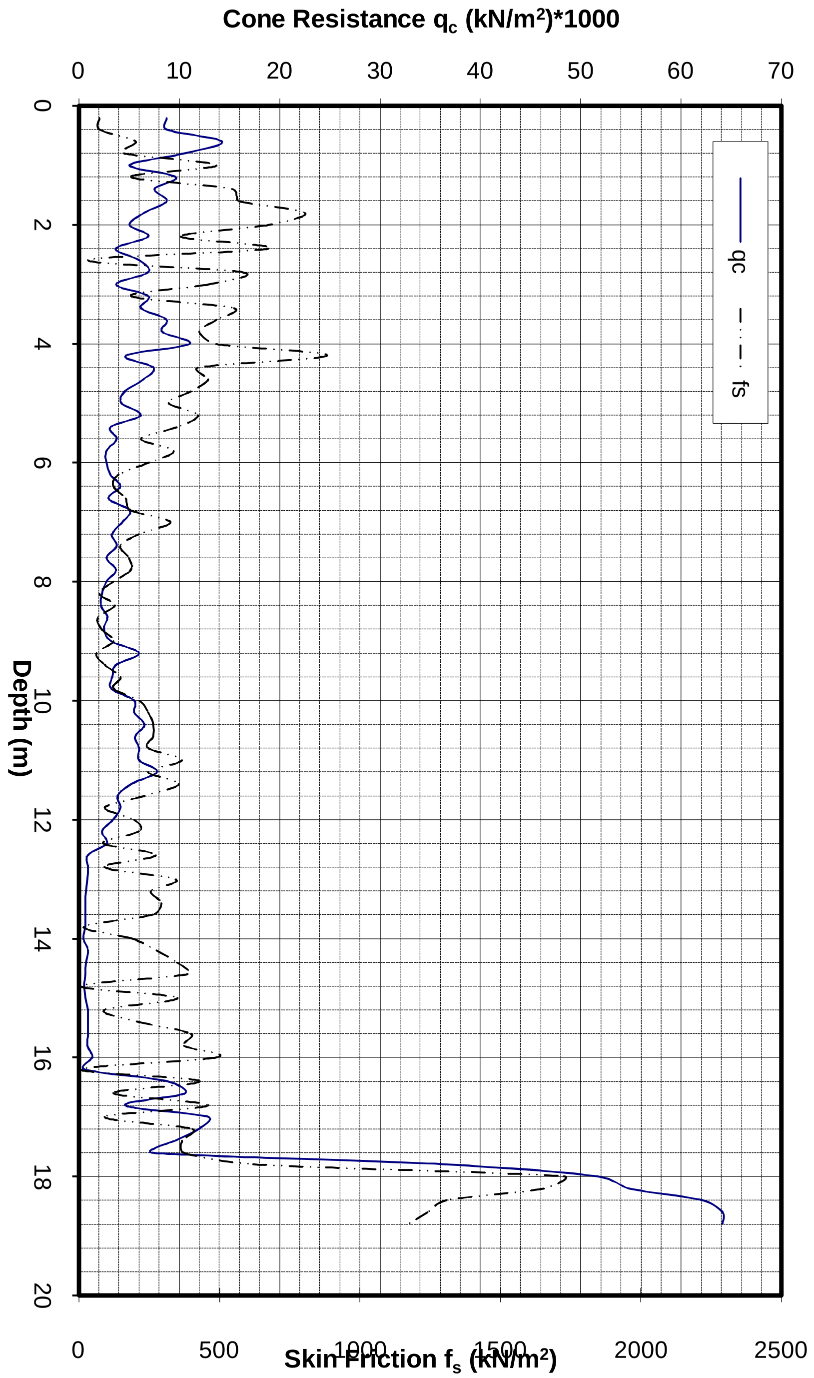
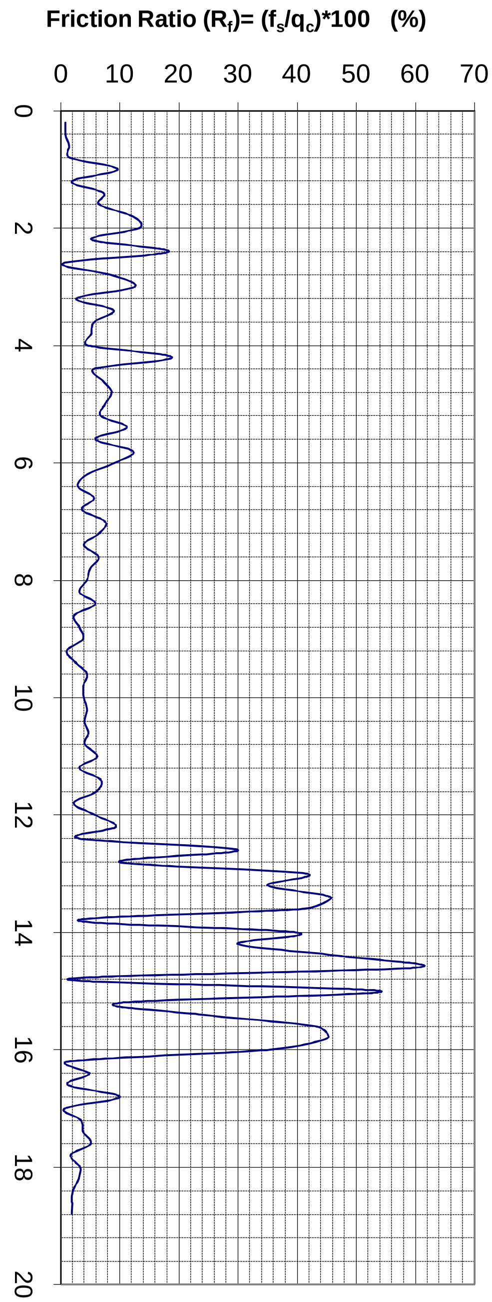
University of Khartoum

Cone Penetration Test Results

Embankment

5





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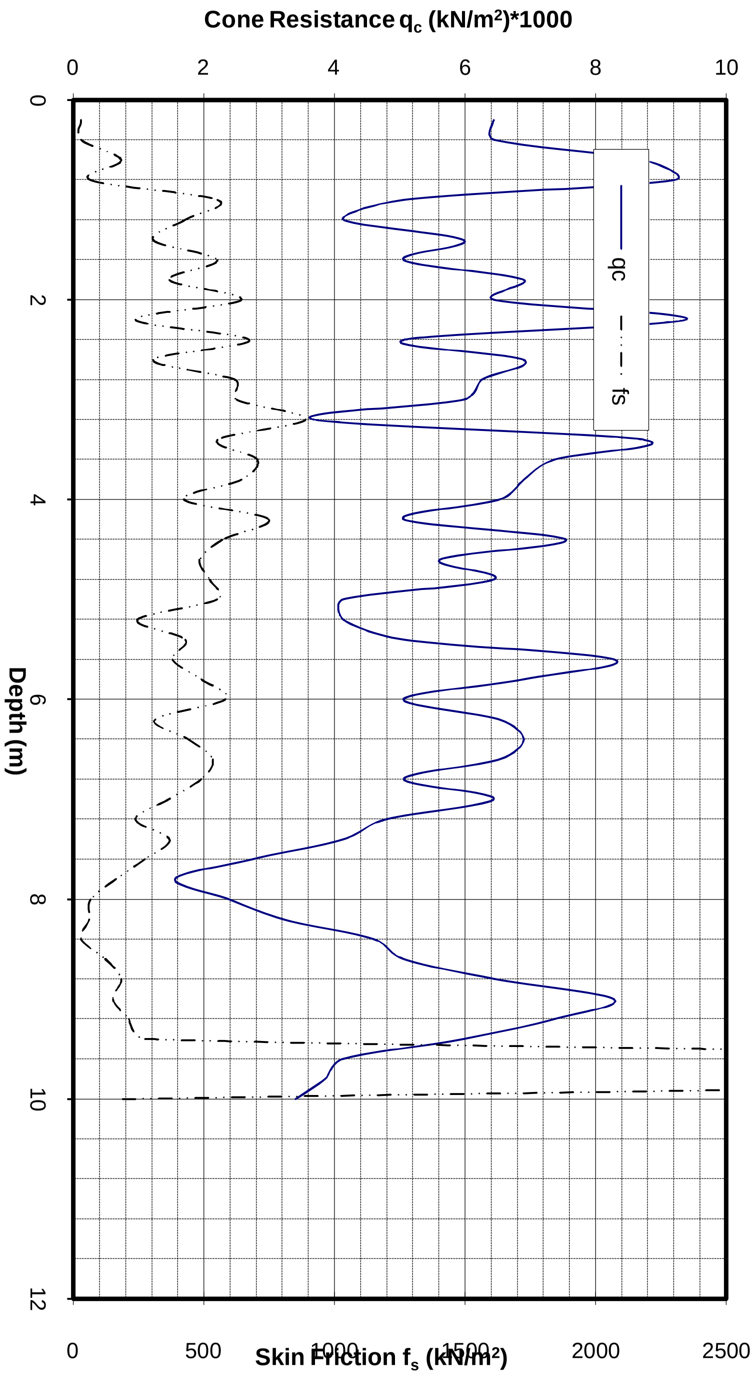
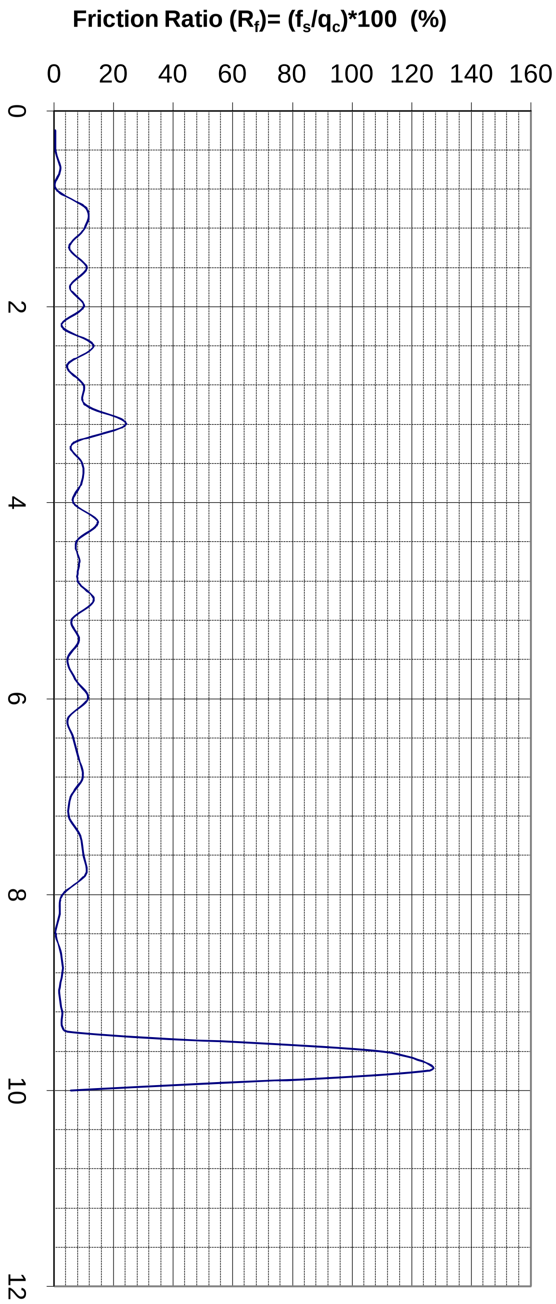
Cone Penetration Test Results

Embankment

6



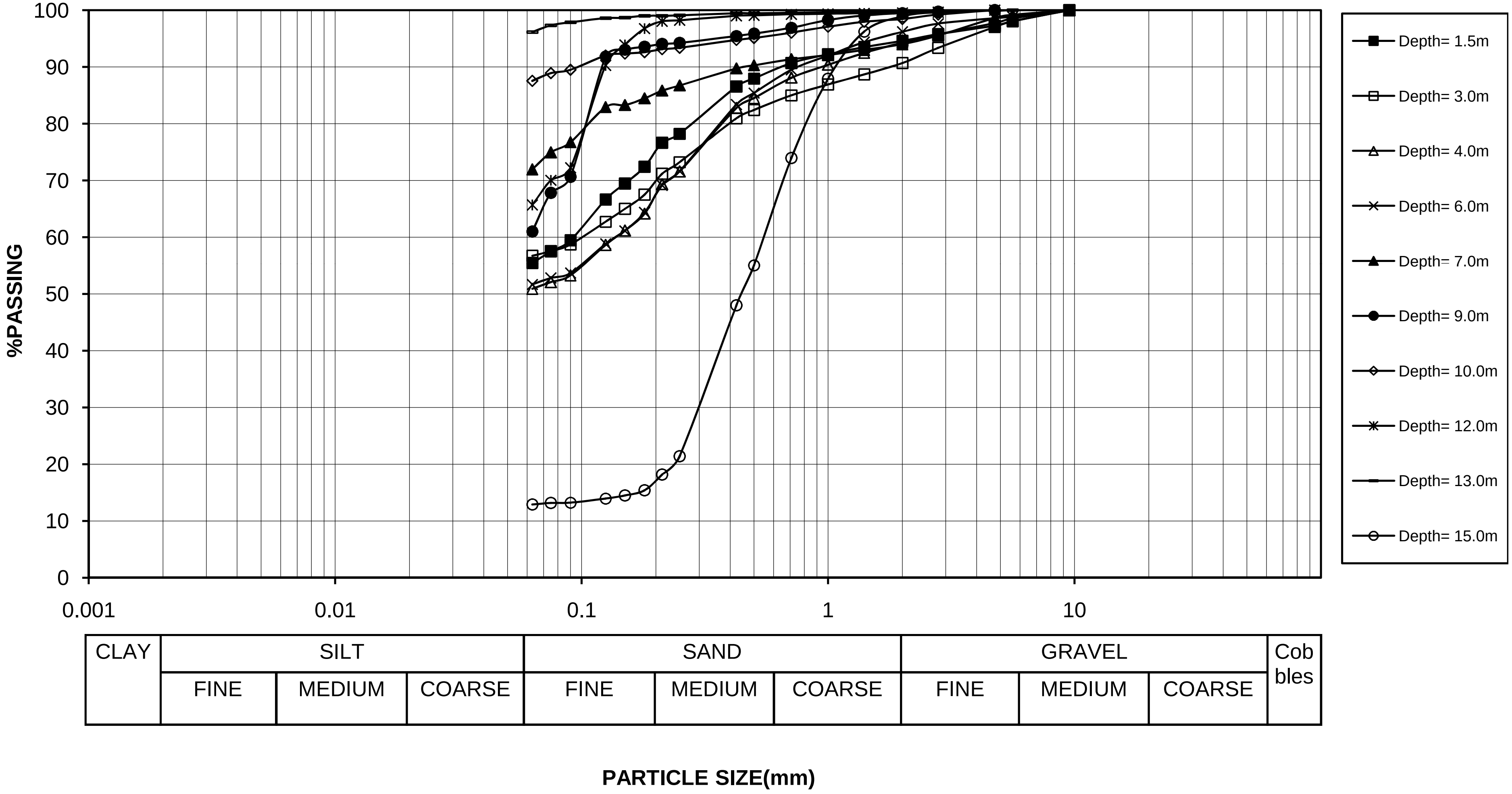


APPENDIX (D)

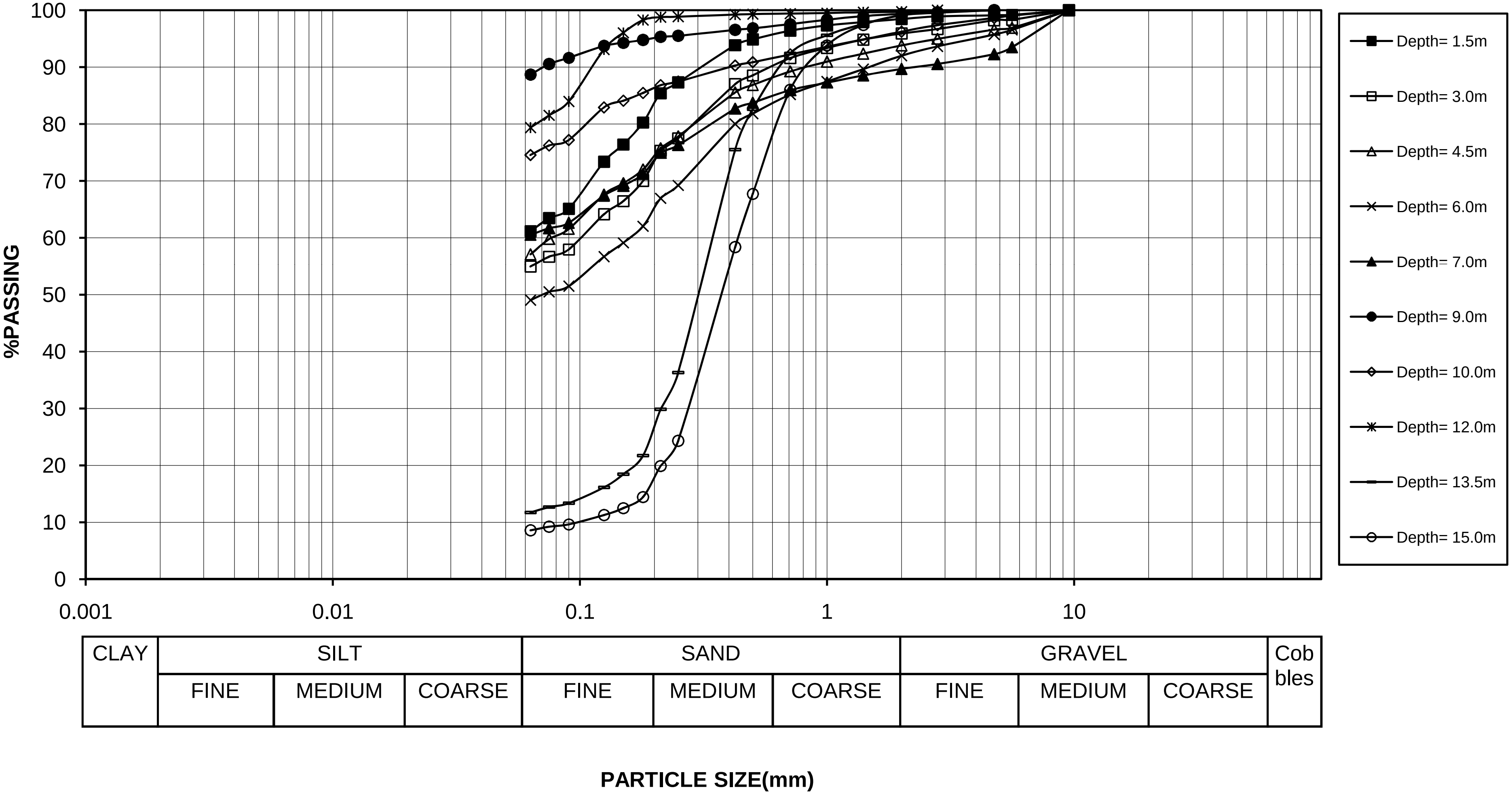
GRAIN SIZE DISTRIBUTION TEST RESULTS

(WET SIEVING – WET SIEVING AND HYDROMETER TEST)

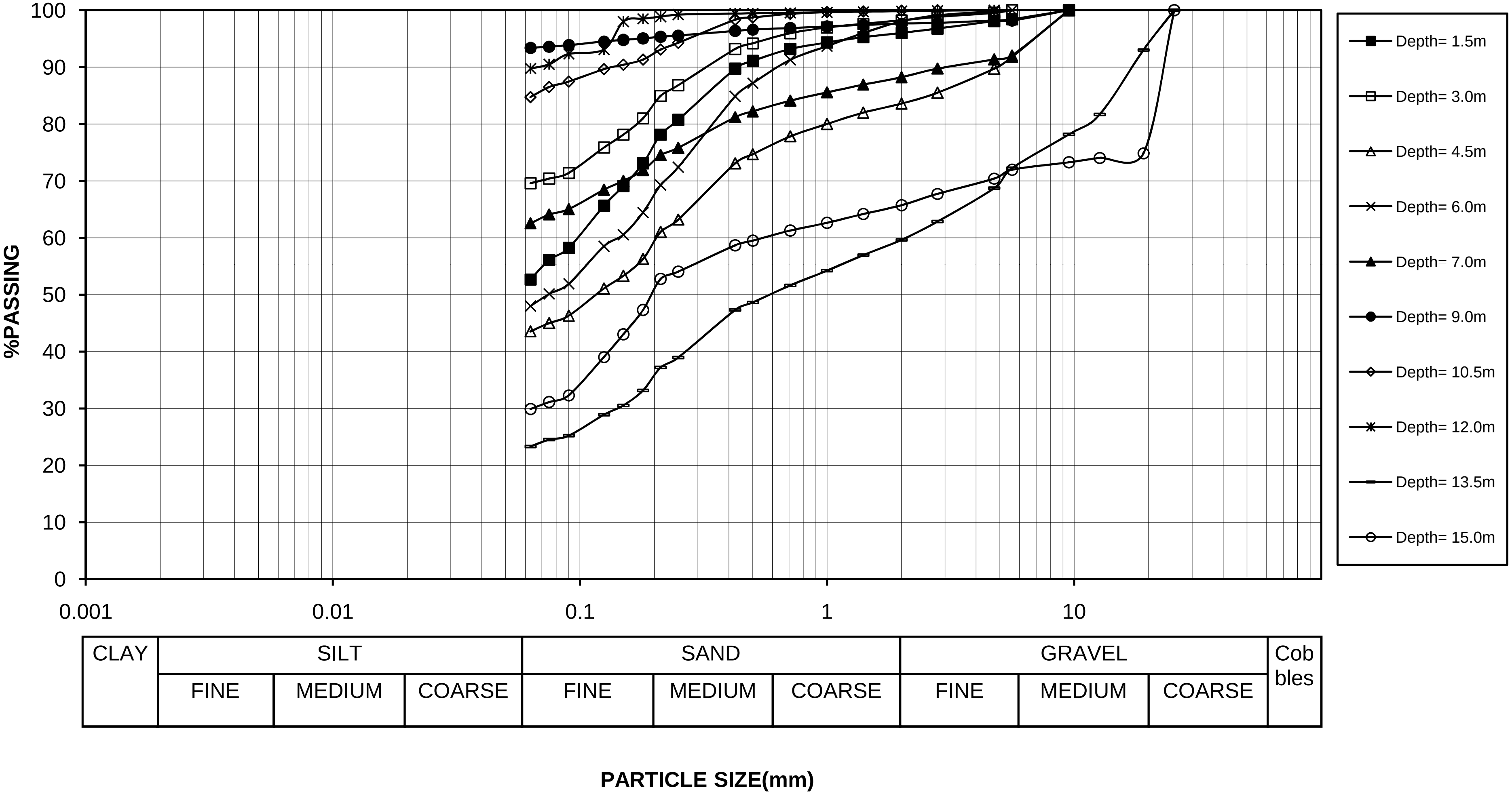
Building and Road Research Institute
University of Khartoum
Embankment
Grain Size Distribution Curves For BH# (1)



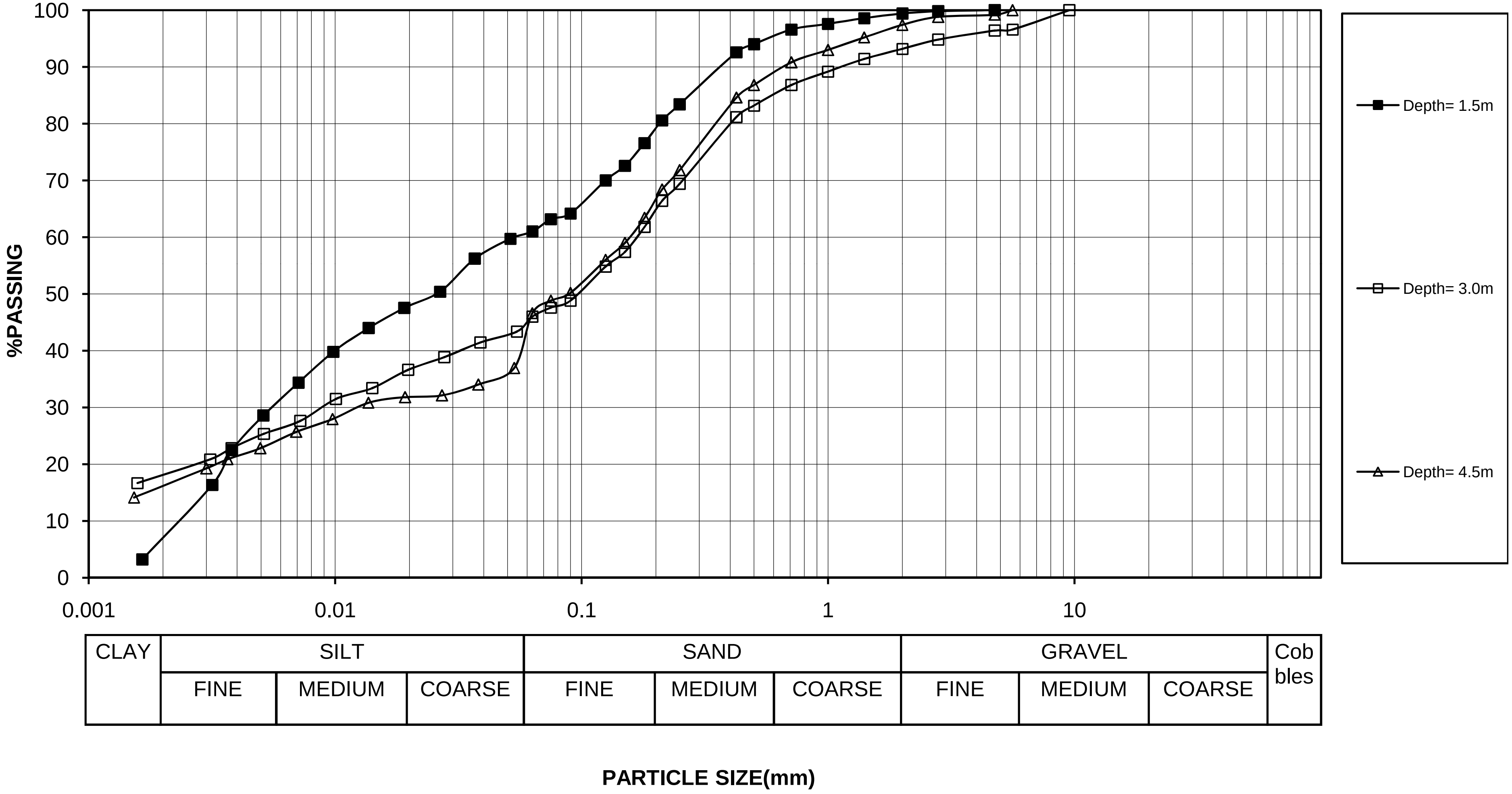
Building and Road Research Institute
University of Khartoum
Embankment
Grain Size Distribution Curves For BH# (2)



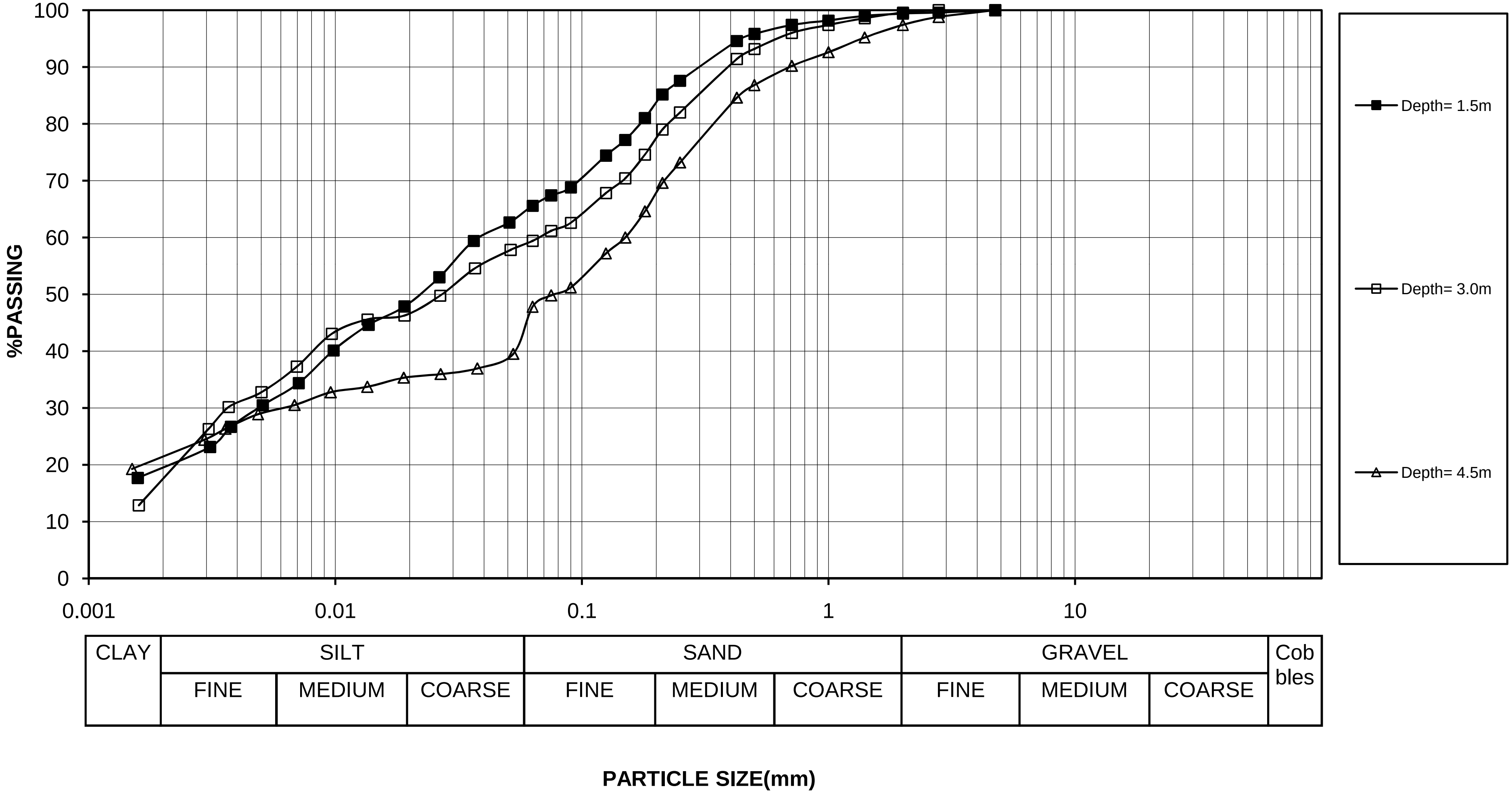
Building and Road Research Institute
University of Khartoum
Embankment
Grain Size Distribution Curves For BH# (3)



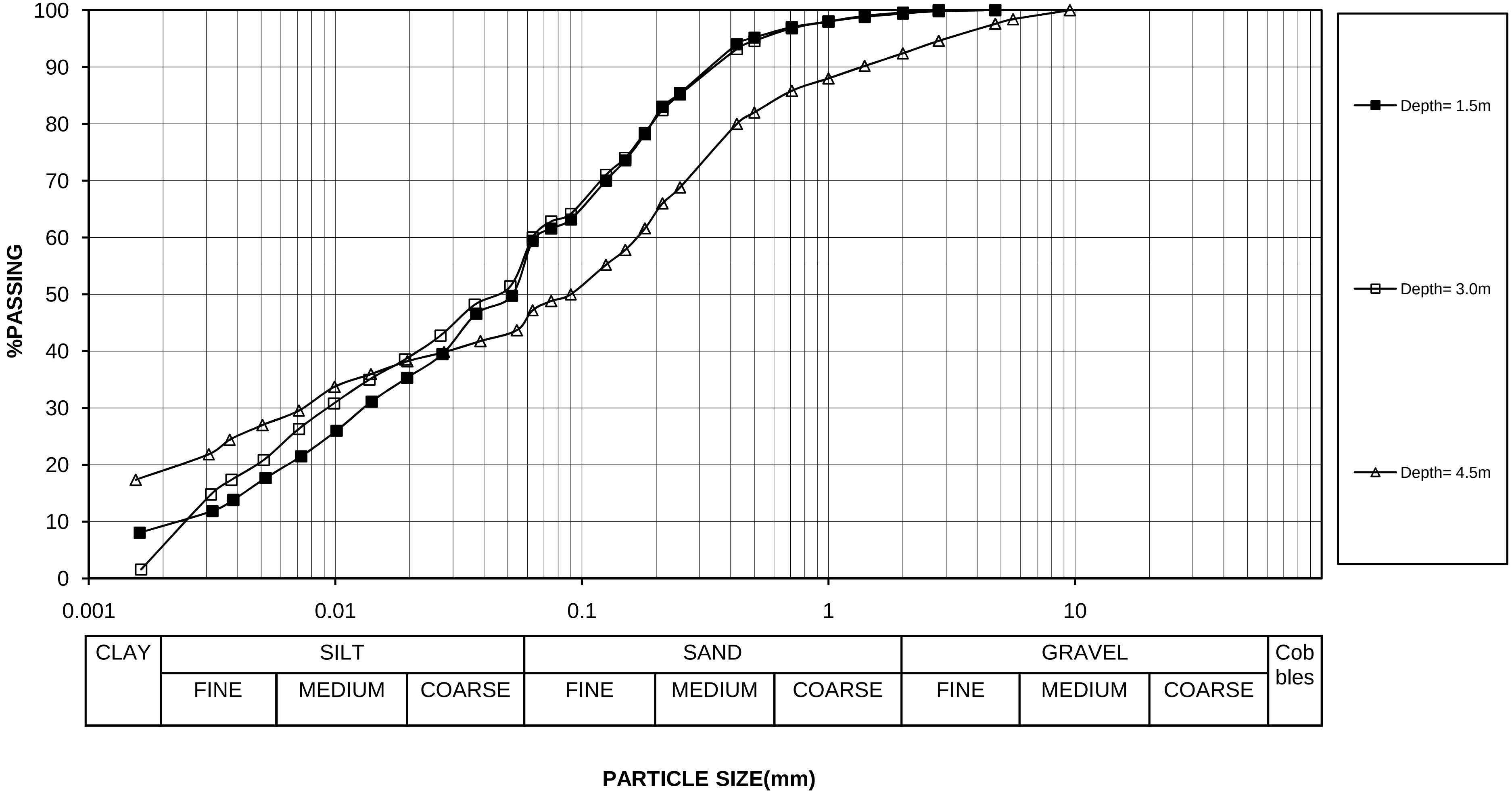
Building and Road Research Institute
University of Khartoum
Embankment
Seive+Hydrometer For BH# (1)



Building and Road Research Institute
University of Khartoum
Embankment
Seive+Hydrometer For BH# (2)



Building and Road Research Institute
University of Khartoum
Embankment
Seive+Hydrometer For BH# (3)



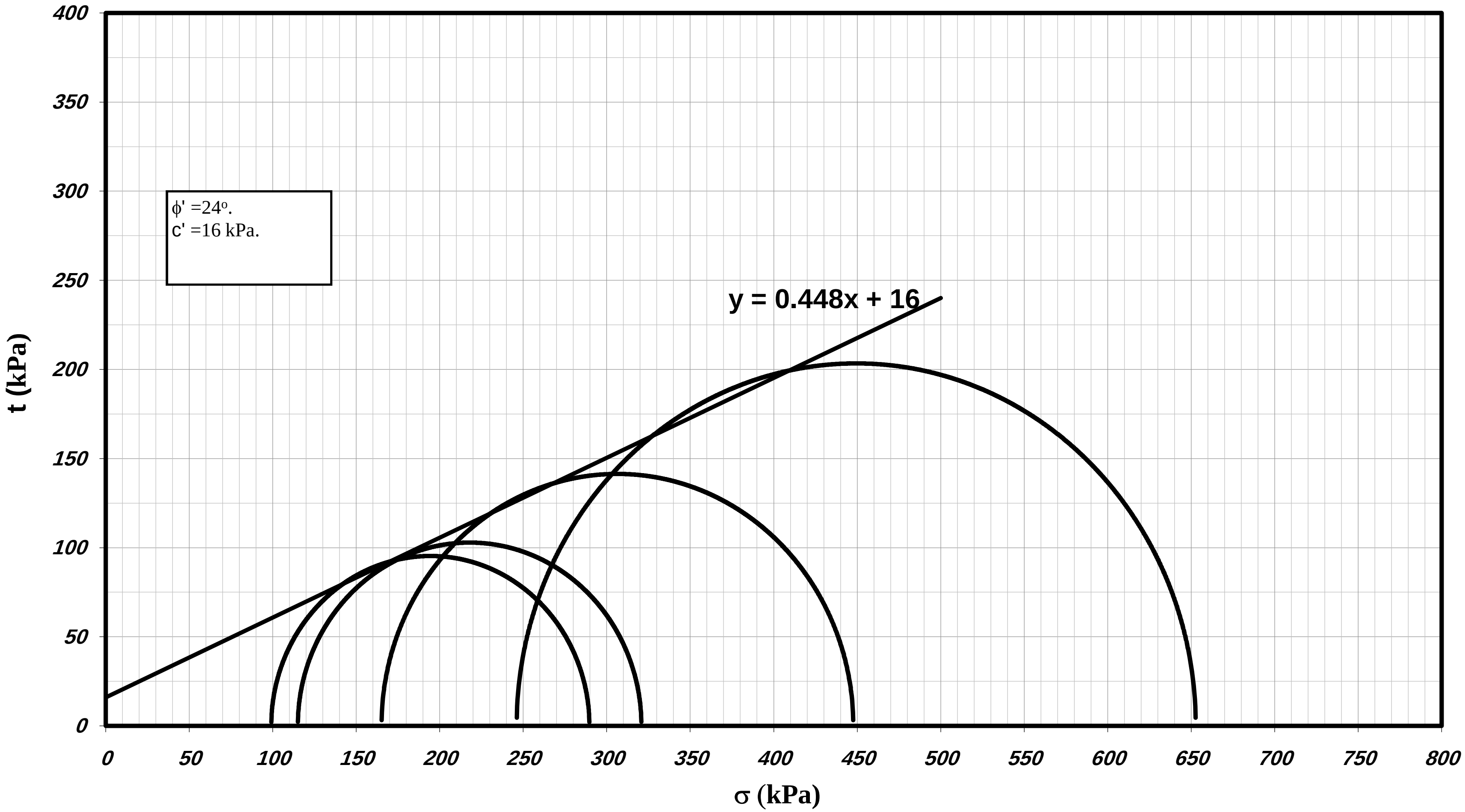
APPENDIX (E)

CONSOLIDATION UNDRAINED (CU) TRIAXIAL

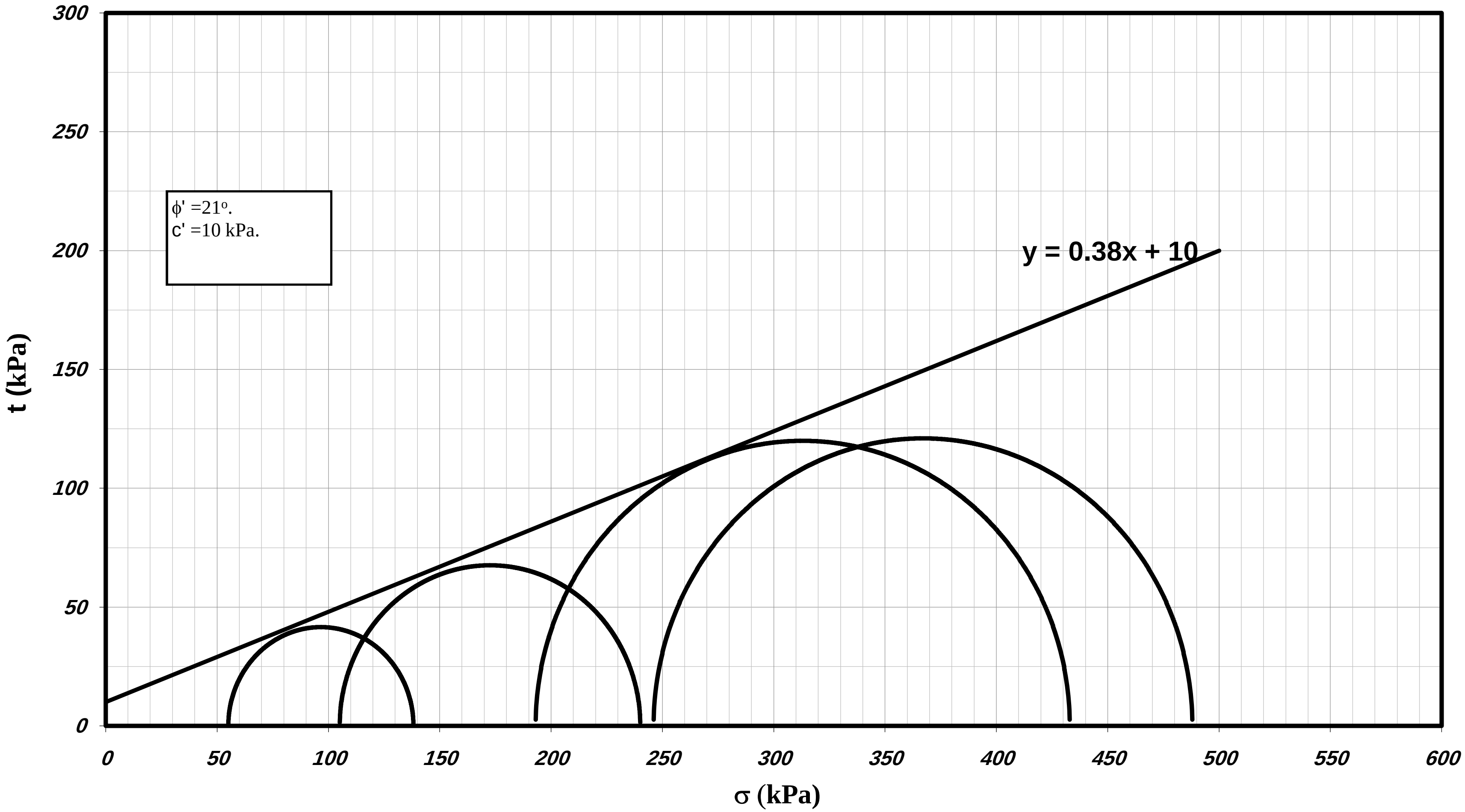
COMPRESSION TEST RESULTS

(MOHR'S CIRCLE AND SHEAR FAILURE ENVELOPE)

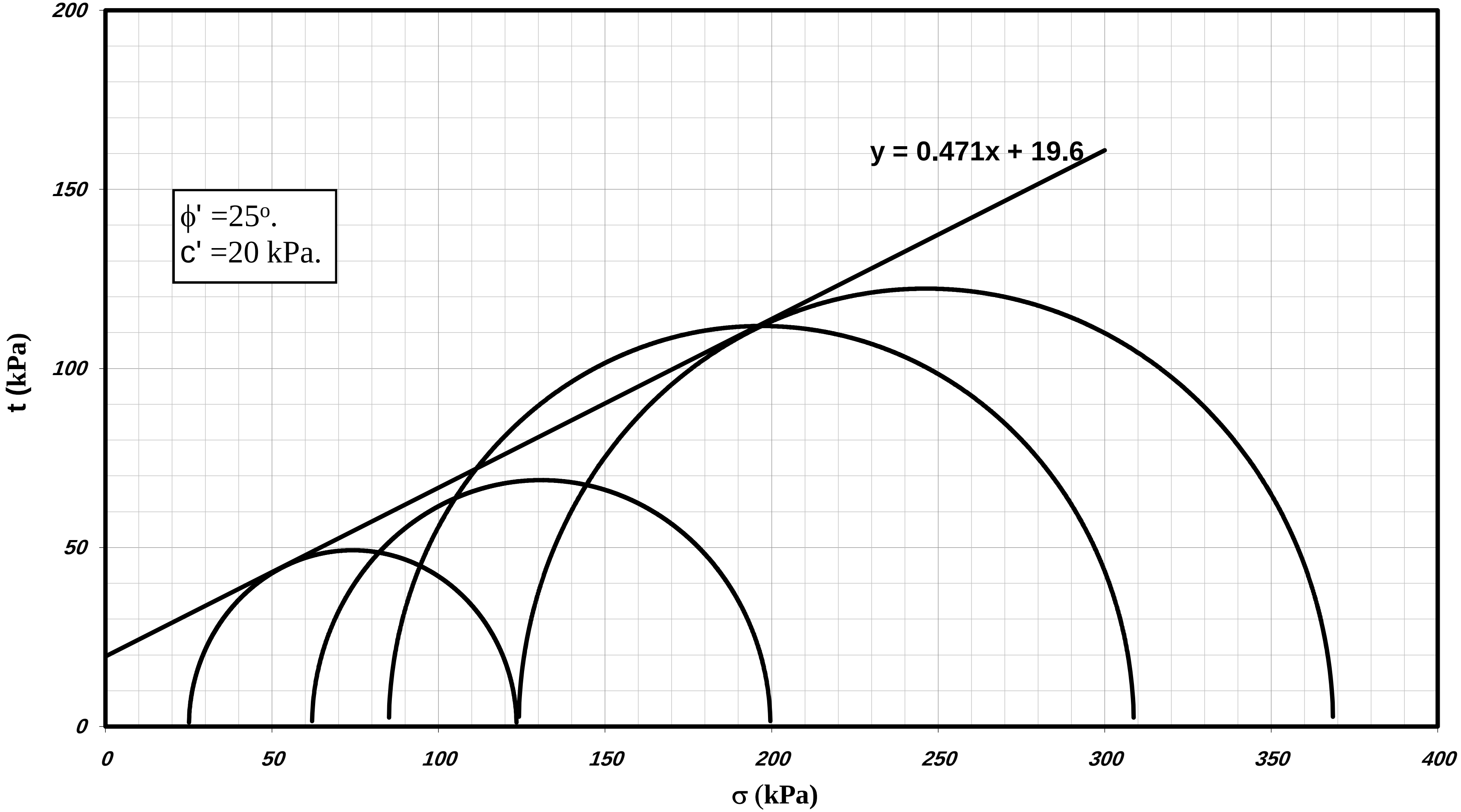
Building and Road Research Institute
University of Khartoum
Embankment
Mohr Circles for BH# 1 Depth :1.50m



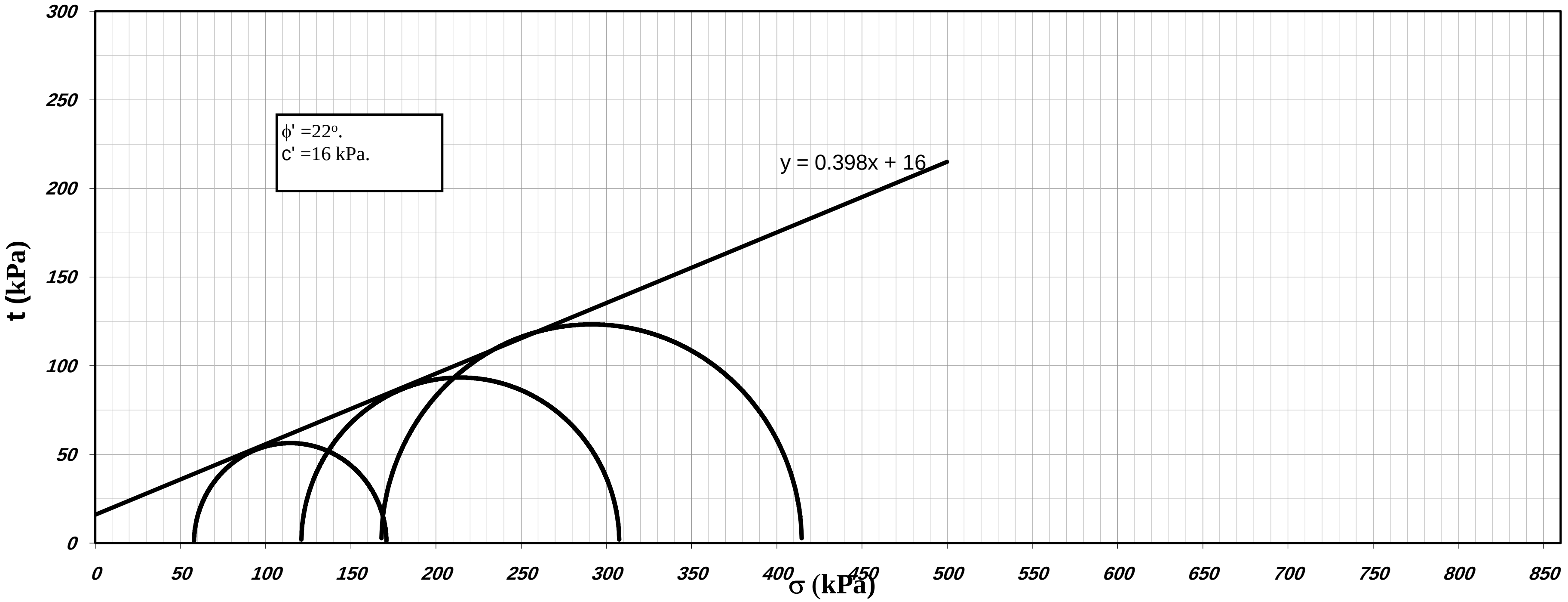
Building and Road Research Institute
University of Khartoum
Embankment
Mohr Circles for BH# 1 Depth :4.50m



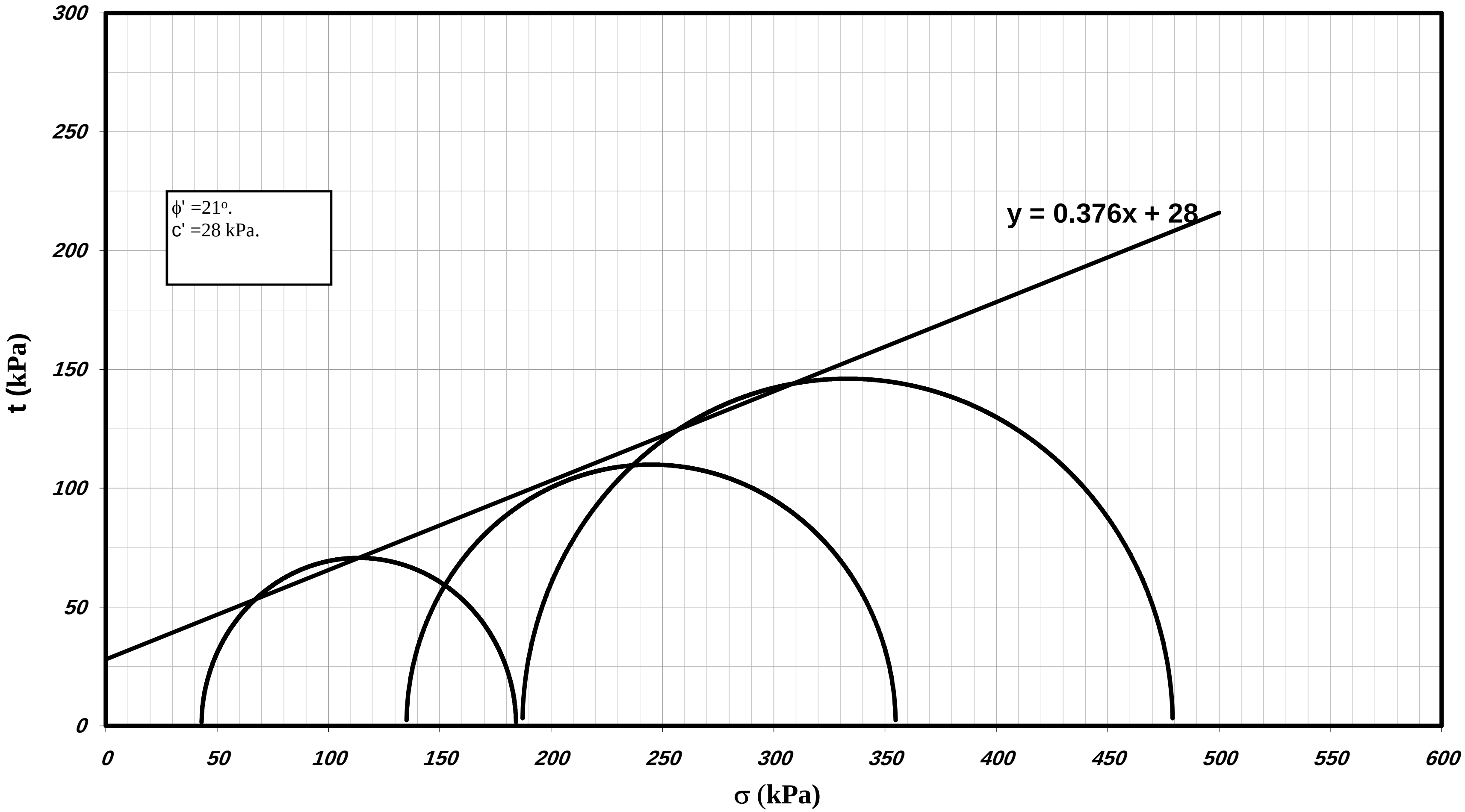
Building and Road Research Institute
University of Khartoum
Embankment
Mohr Circles for BH# 1 Depth :7.50m



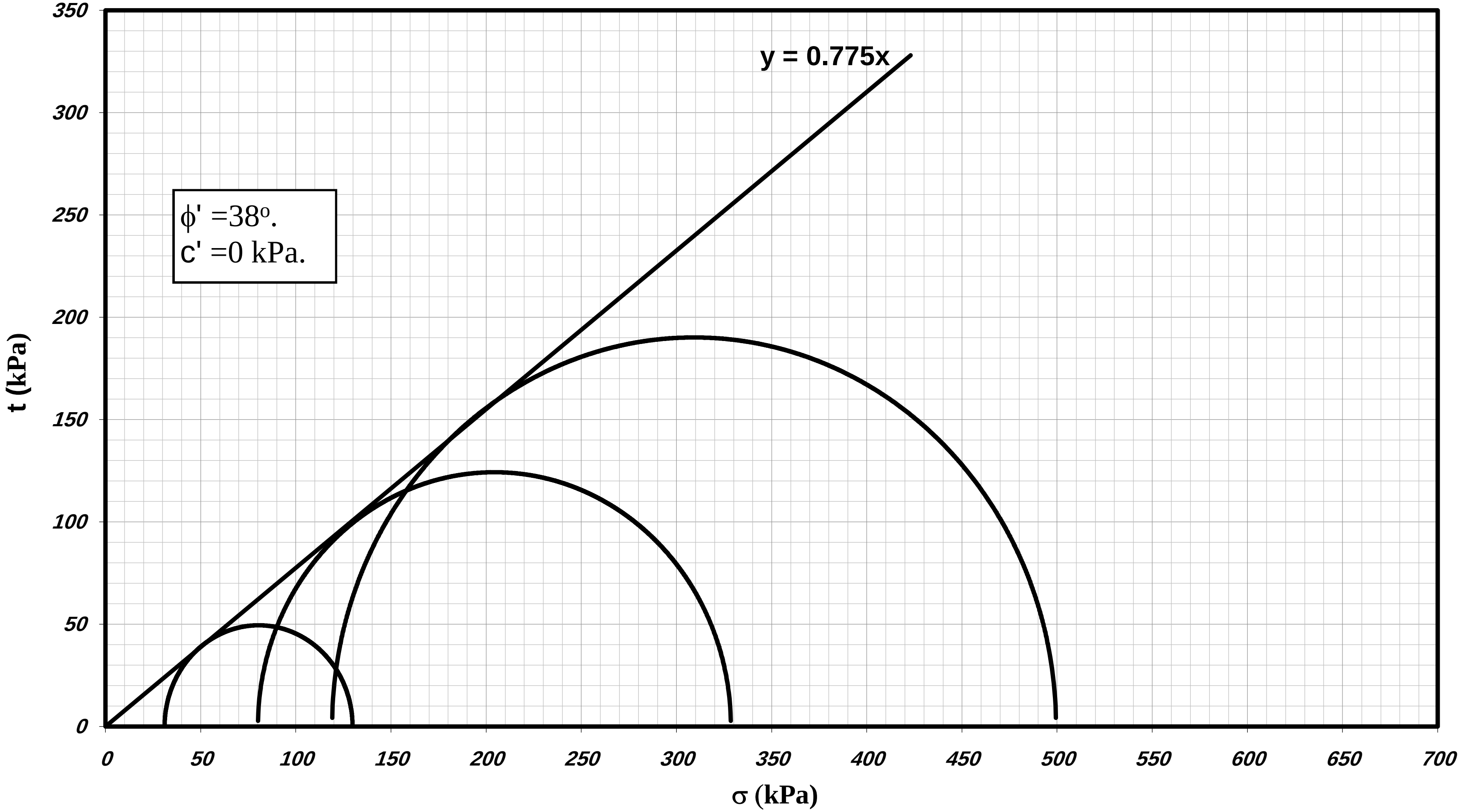
Building and Road Research Institute
University of Khartoum
Embankment
Mohr Circles for BH# 2 Depth :3.0m



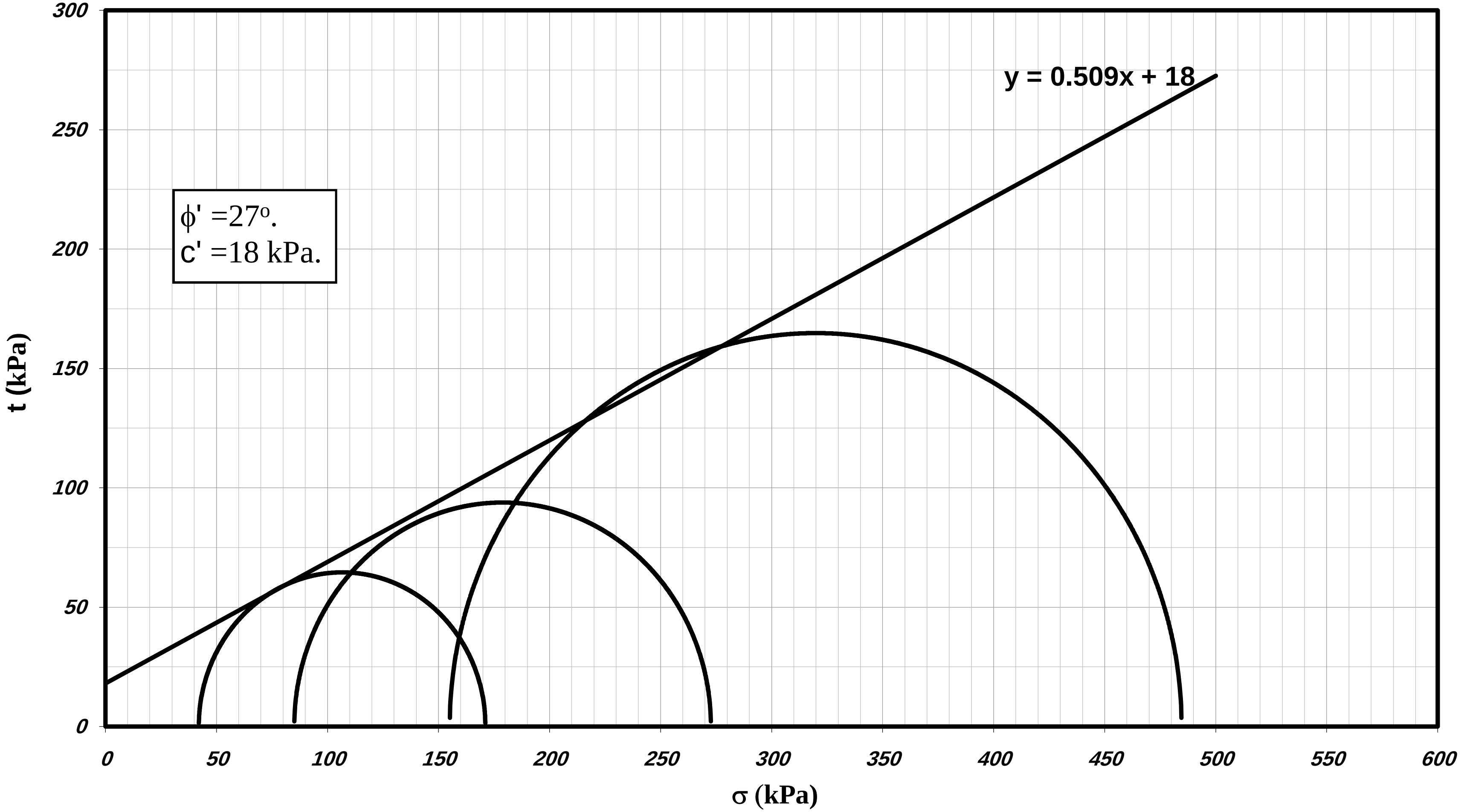
Building and Road Research Institute
University of Khartoum
Embankment
Mohr Circles for BH# 2 Depth :6.0m



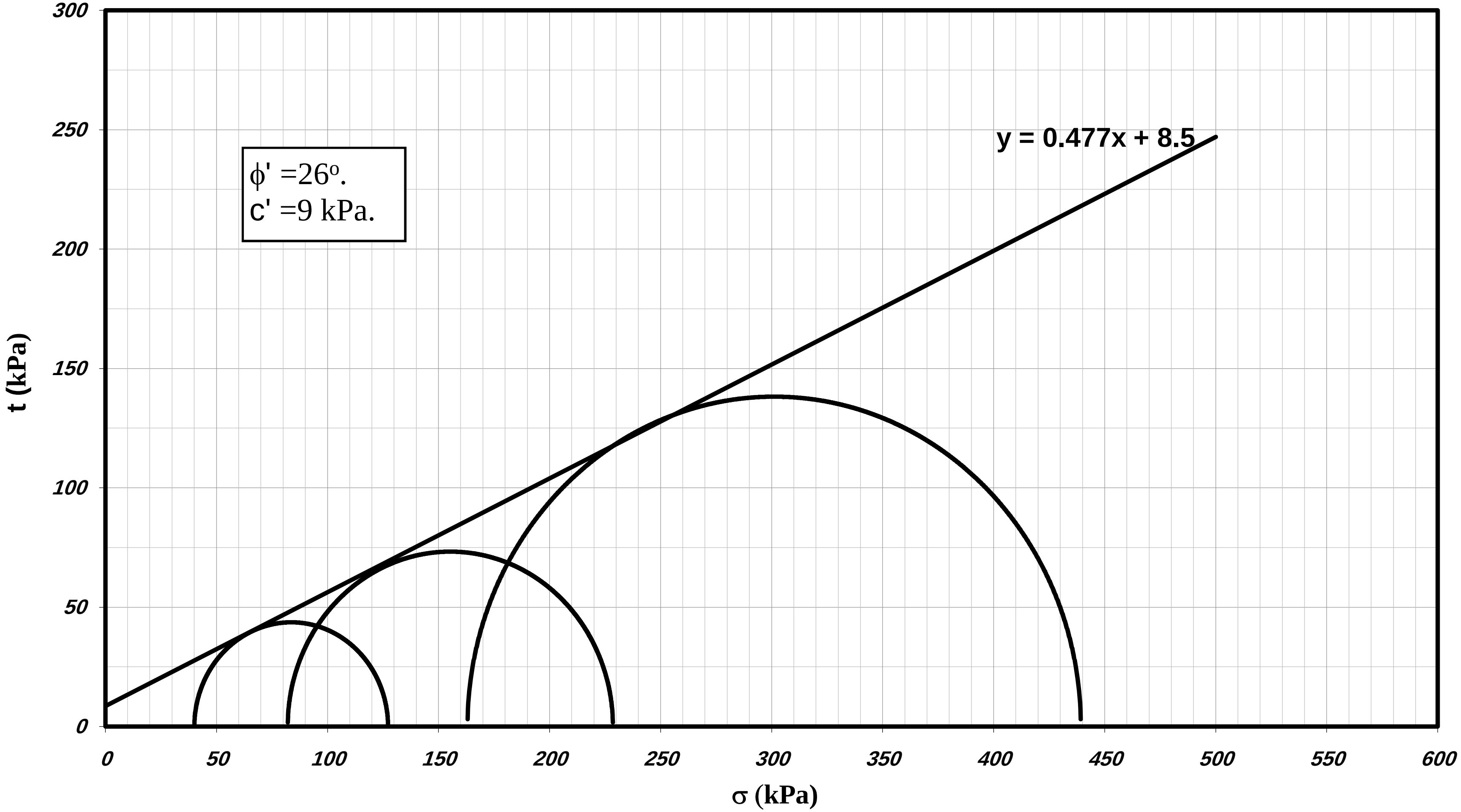
Building and Road Research Institute
University of Khartoum
Embankment
Mohr Circles for BH# 2 Depth :9.0m



Building and Road Research Institute
University of Khartoum
Embankment
Mohr Circles for BH# 3 Depth :3.0m



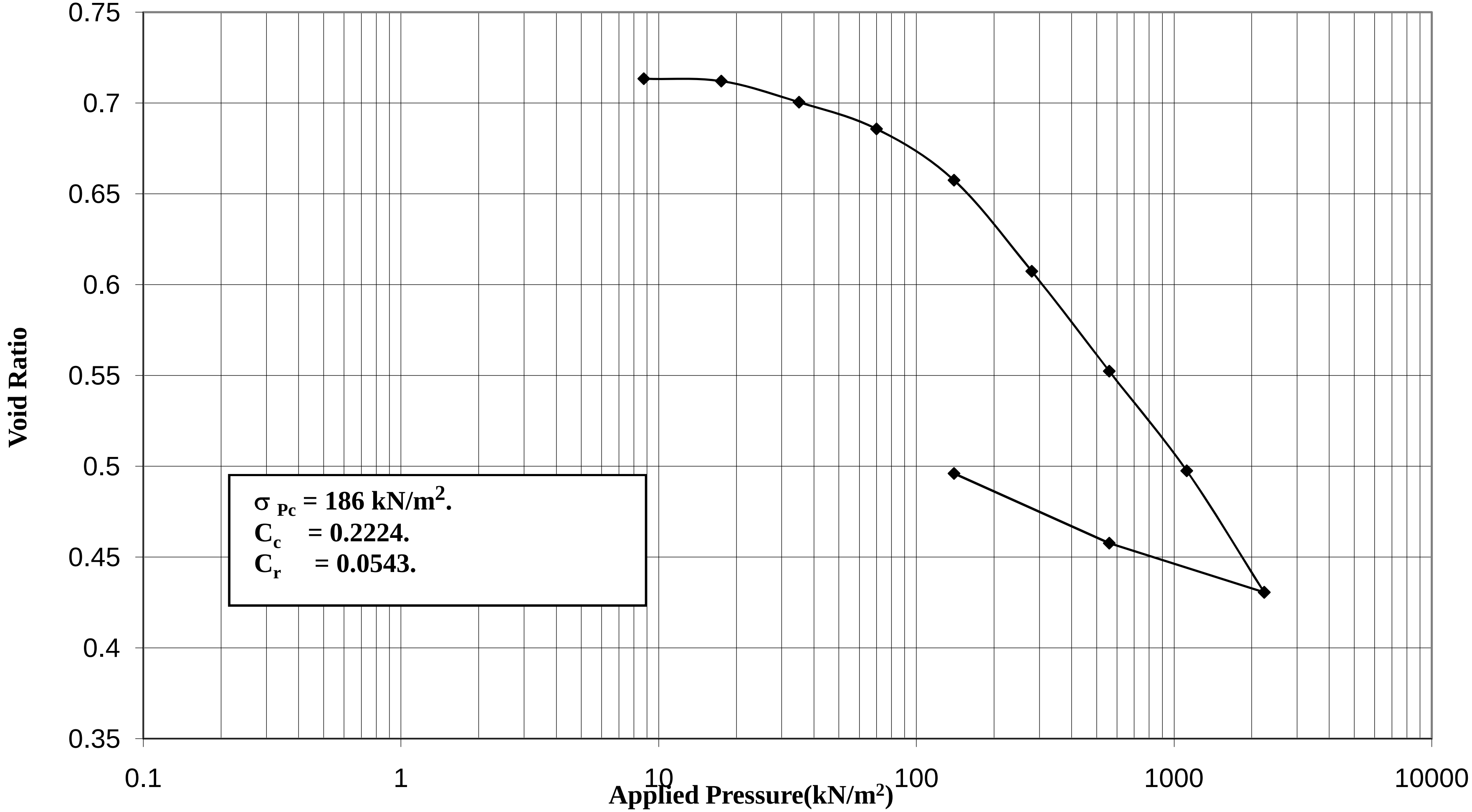
Building and Road Research Institute
University of Khartoum
Embankment
Mohr Circles for BH# 3 Depth :7.5m

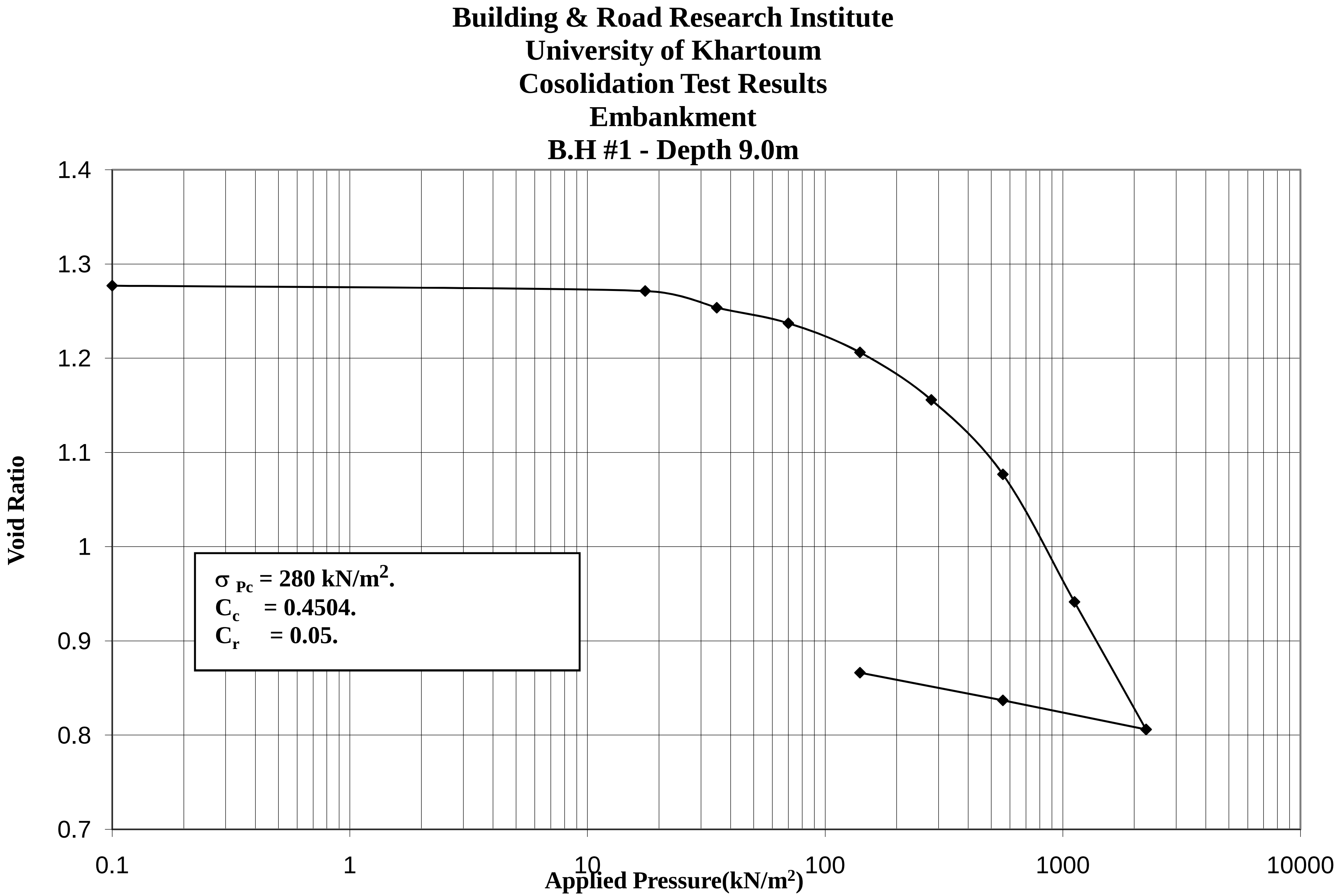


APPENDIX (F)

ONE DIMENTIONAL CONSLIDATION TEST RESULTS

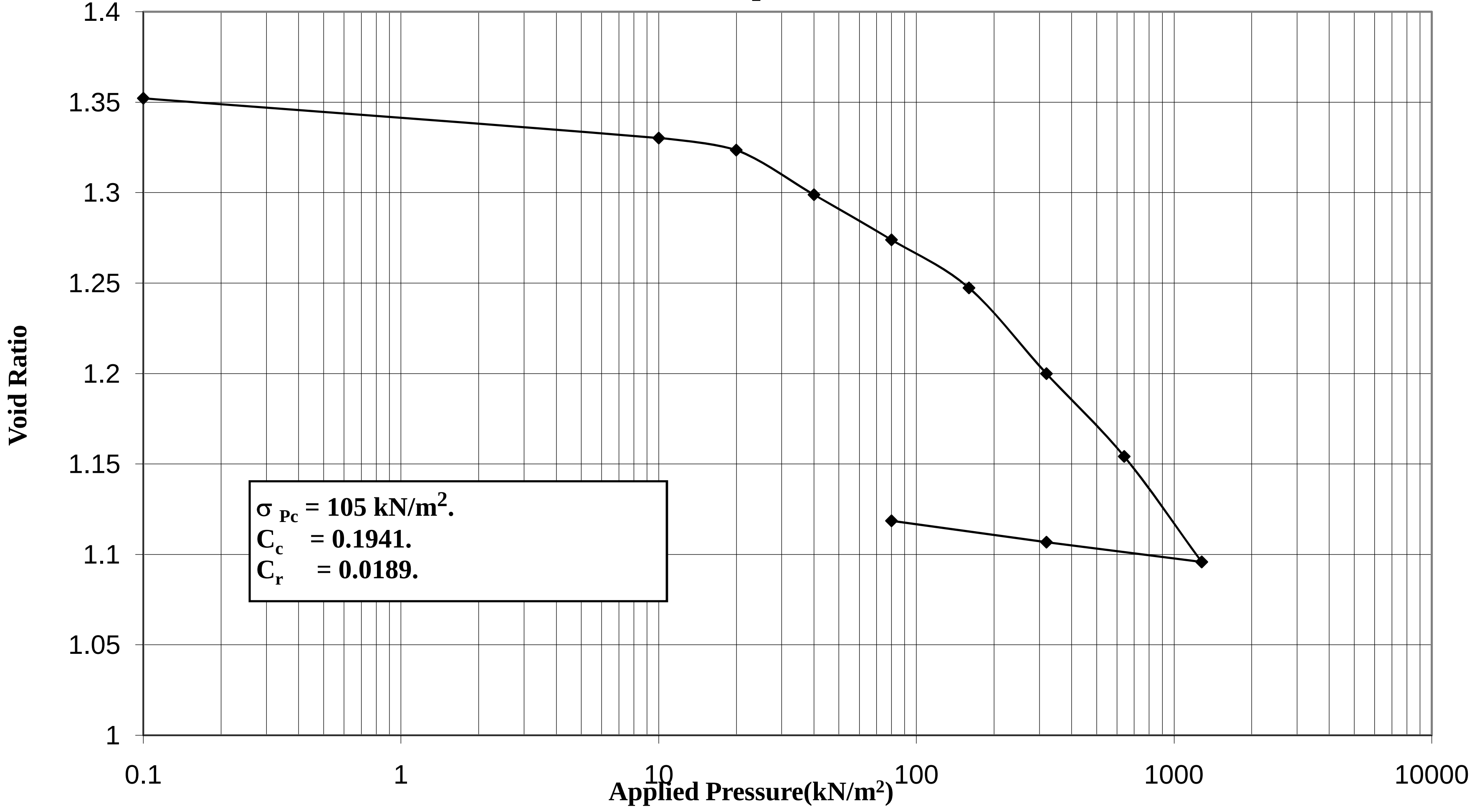
Building & Road Research Institute
University of Khartoum
Cosolidation Test Results
Embankment
B.H #1 - Depth 6.0m



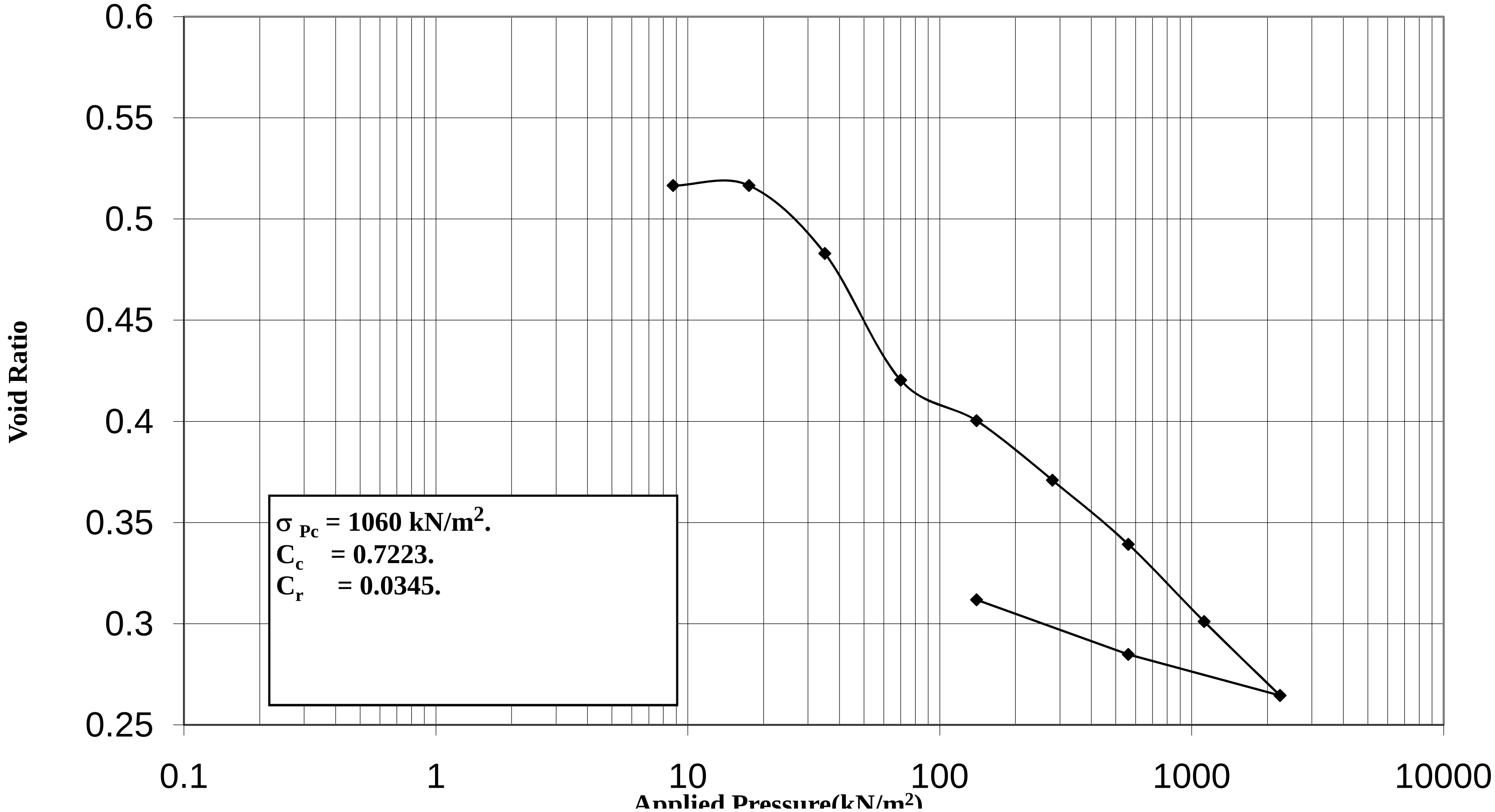


Building & Road Research Institute
University of Khartoum
Cosolidation Test Results
Embankment
B.H #1 - Depth 9.0m
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
Void Ratio
σ Pc = 280 kN/m2.
Cc = 0.4504.
Cr = 0.05.
0.1
1
10
100
1000
10000
Applied Pressure(kN/m2)

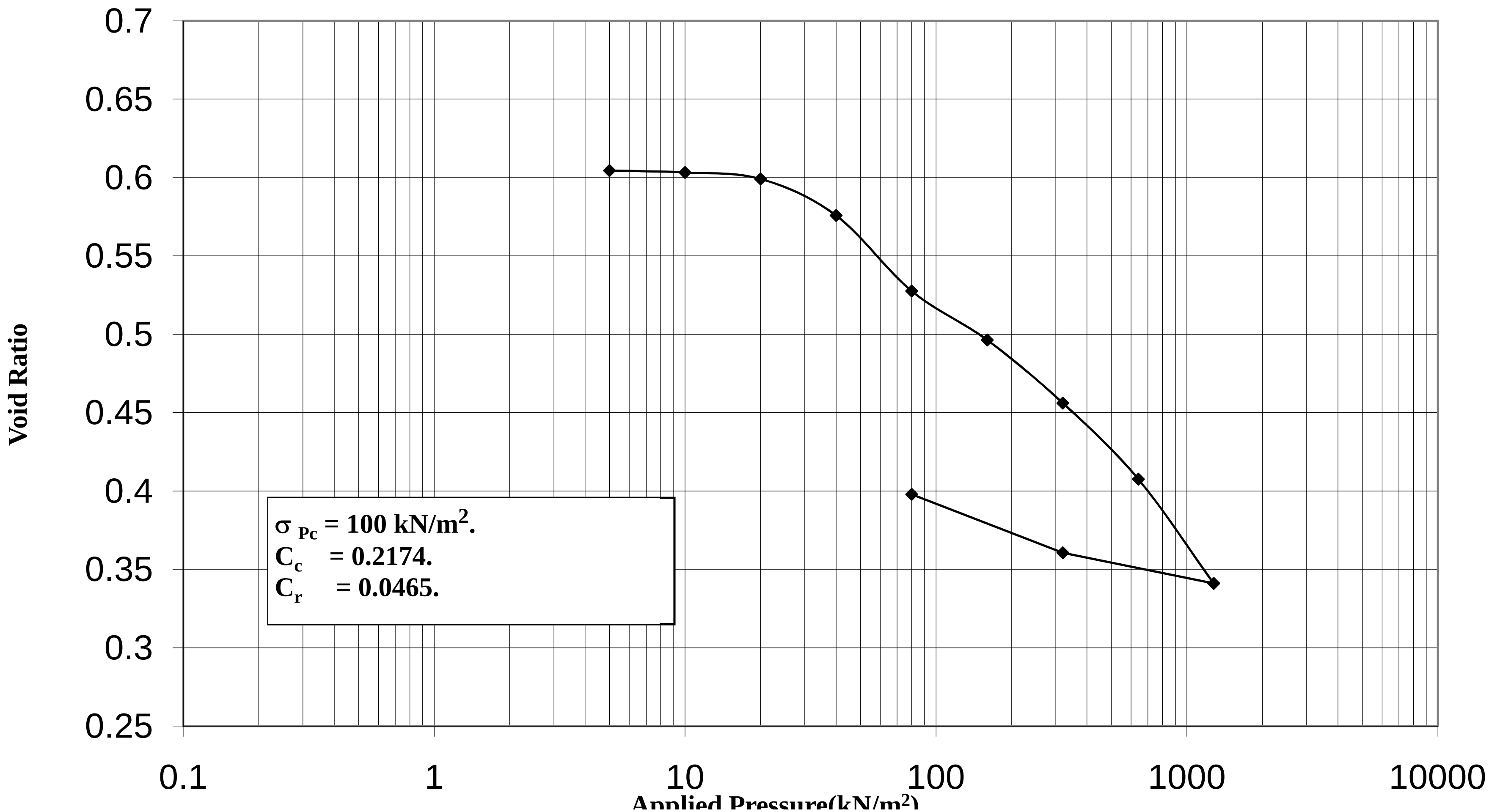
Building & Road Research Institute
University of Khartoum
Cosolidation Test Results
Embankment
B.H #1 - Depth 12.0m



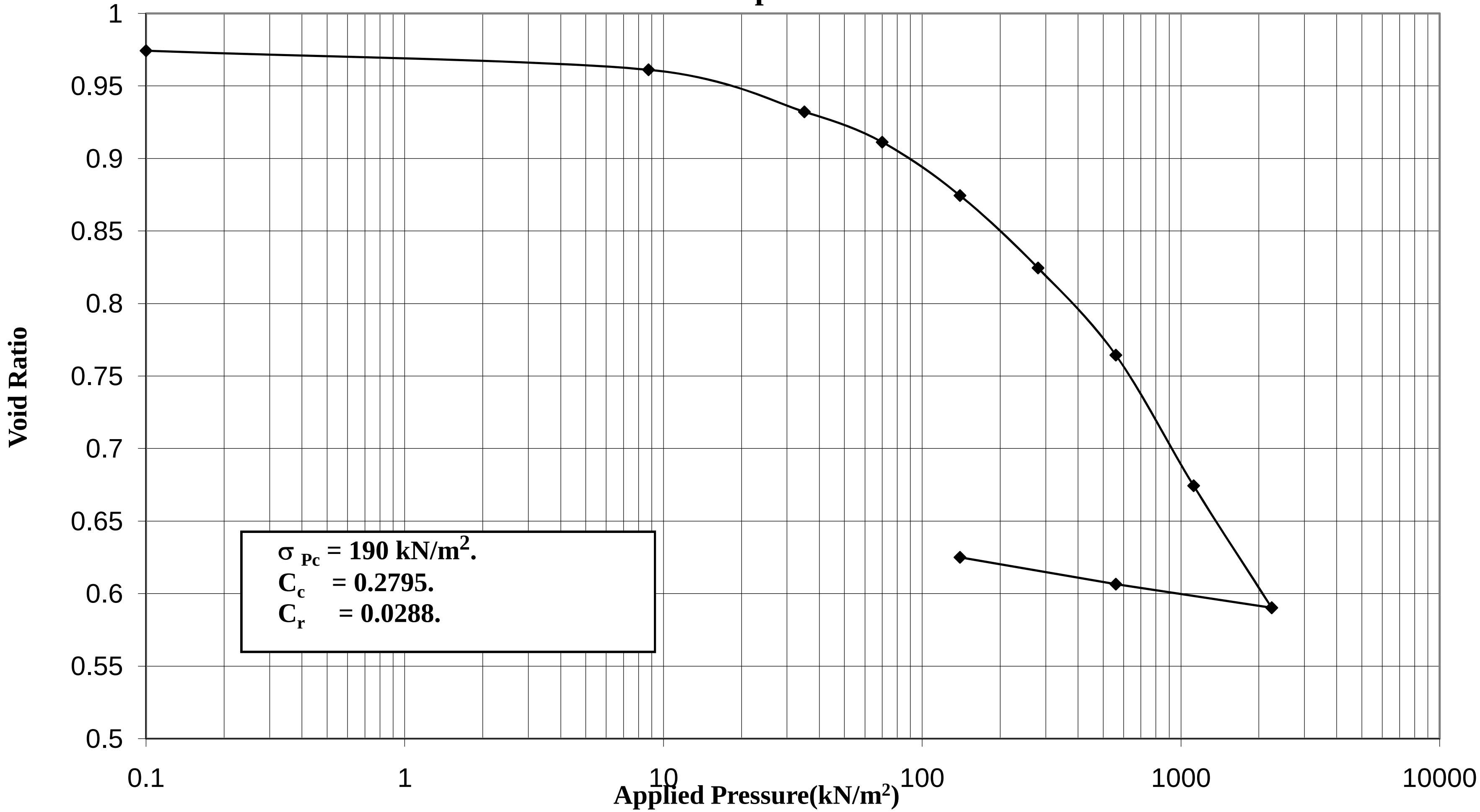
Building & Road Research Institute
University of Khartoum
Cosolidation Test Results
Embankment
B.H #2 - Depth 6.0m



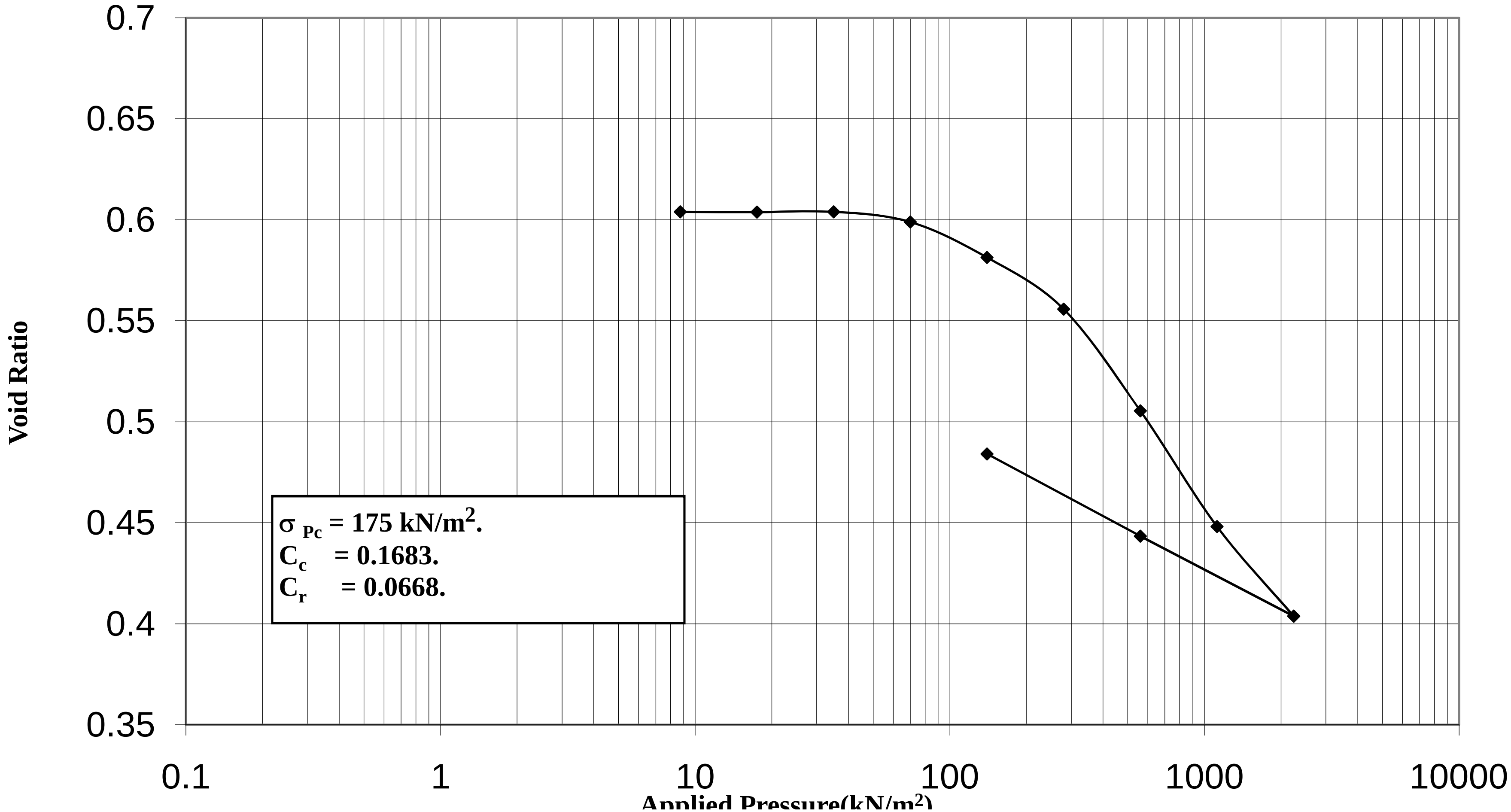
Building & Road Research Institute
University of Khartoum
Cosolidation Test Results
Embankment
B.H #2 - Depth 7.5m



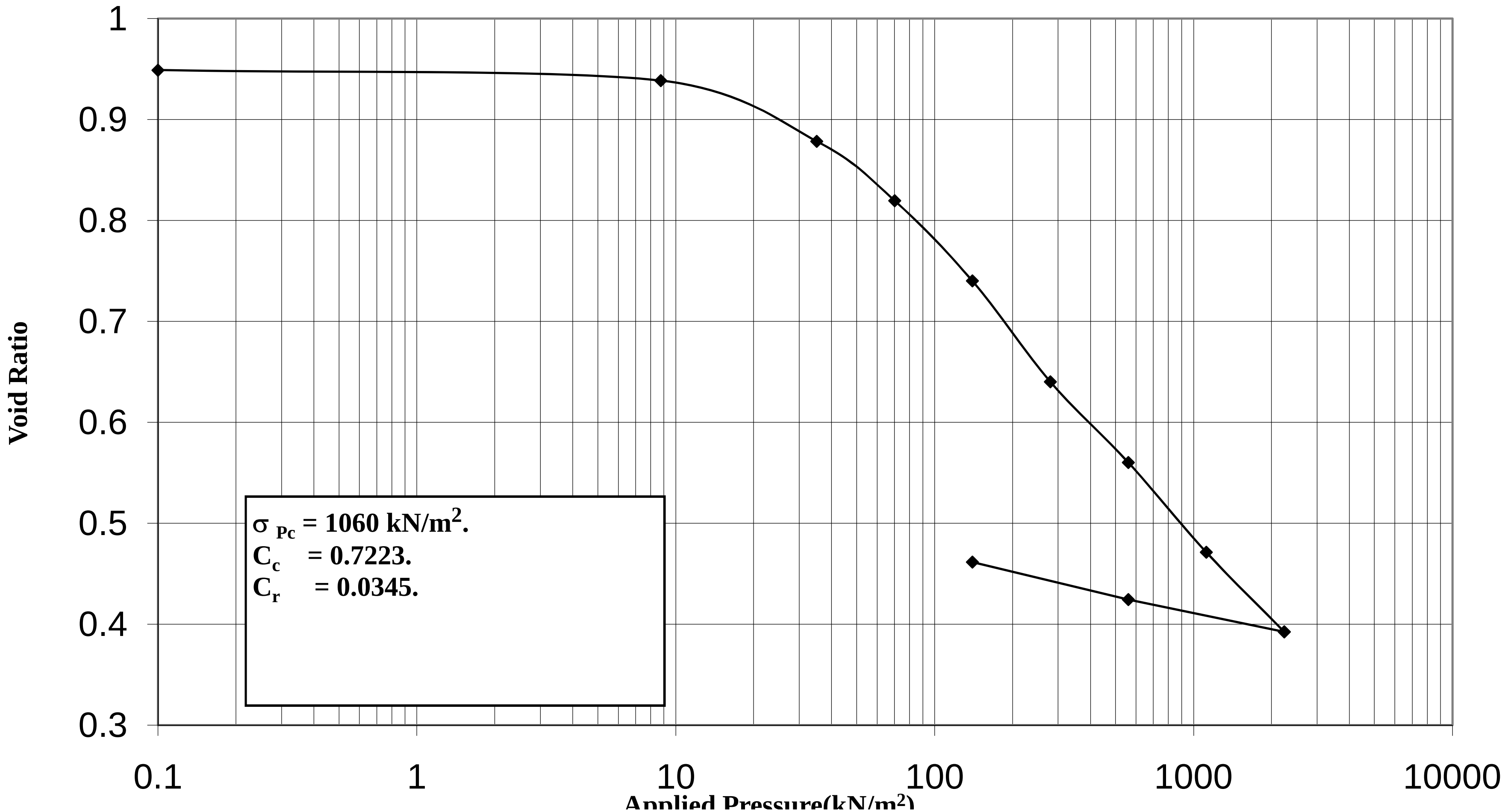
Building & Road Research Institute
University of Khartoum
Cosolidation Test Results
Embankment
B.H #2 - Depth 10.5m

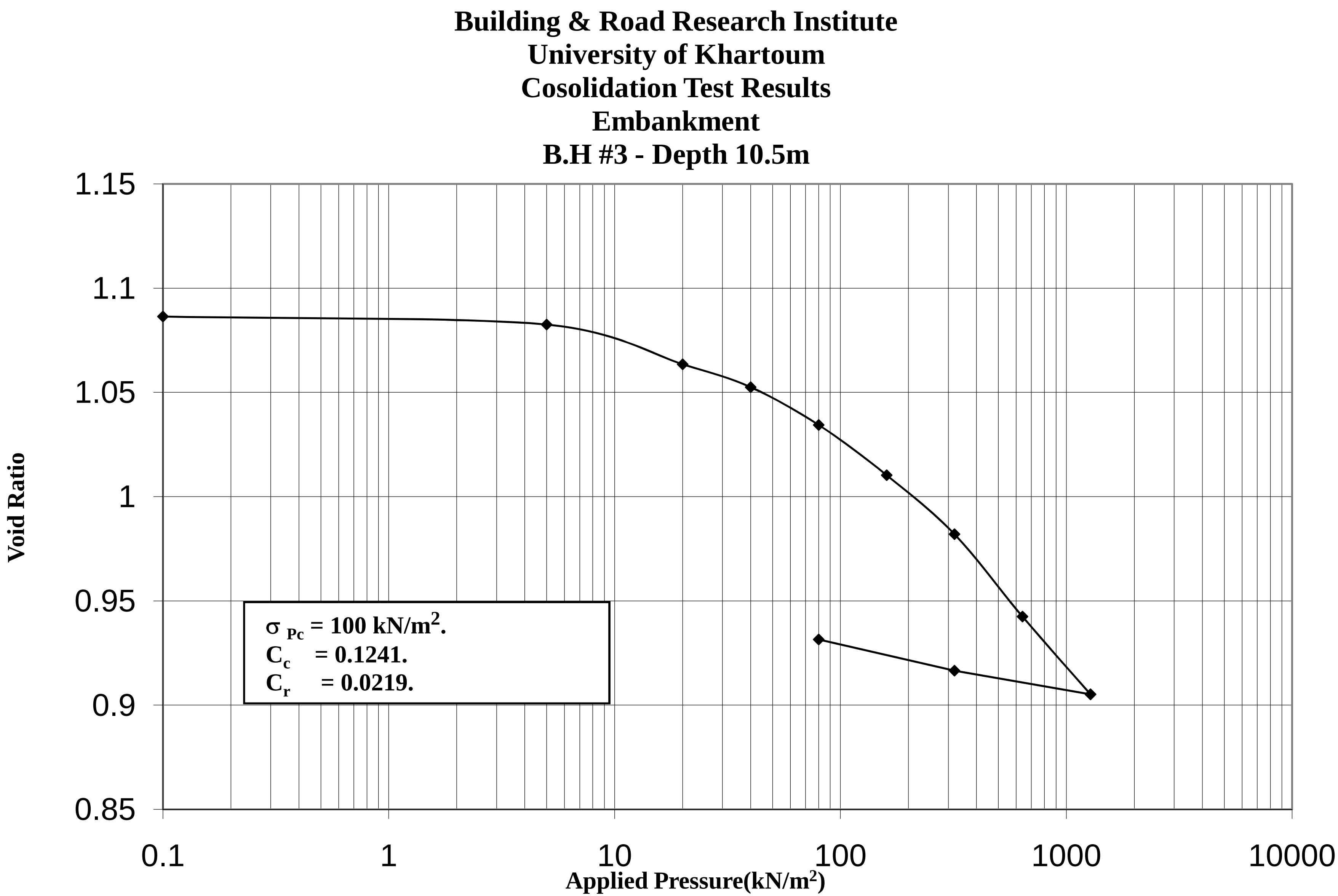


Building & Road Research Institute
University of Khartoum
Cosolidation Test Results
Embankment
B.H #3 - Depth 7.5m



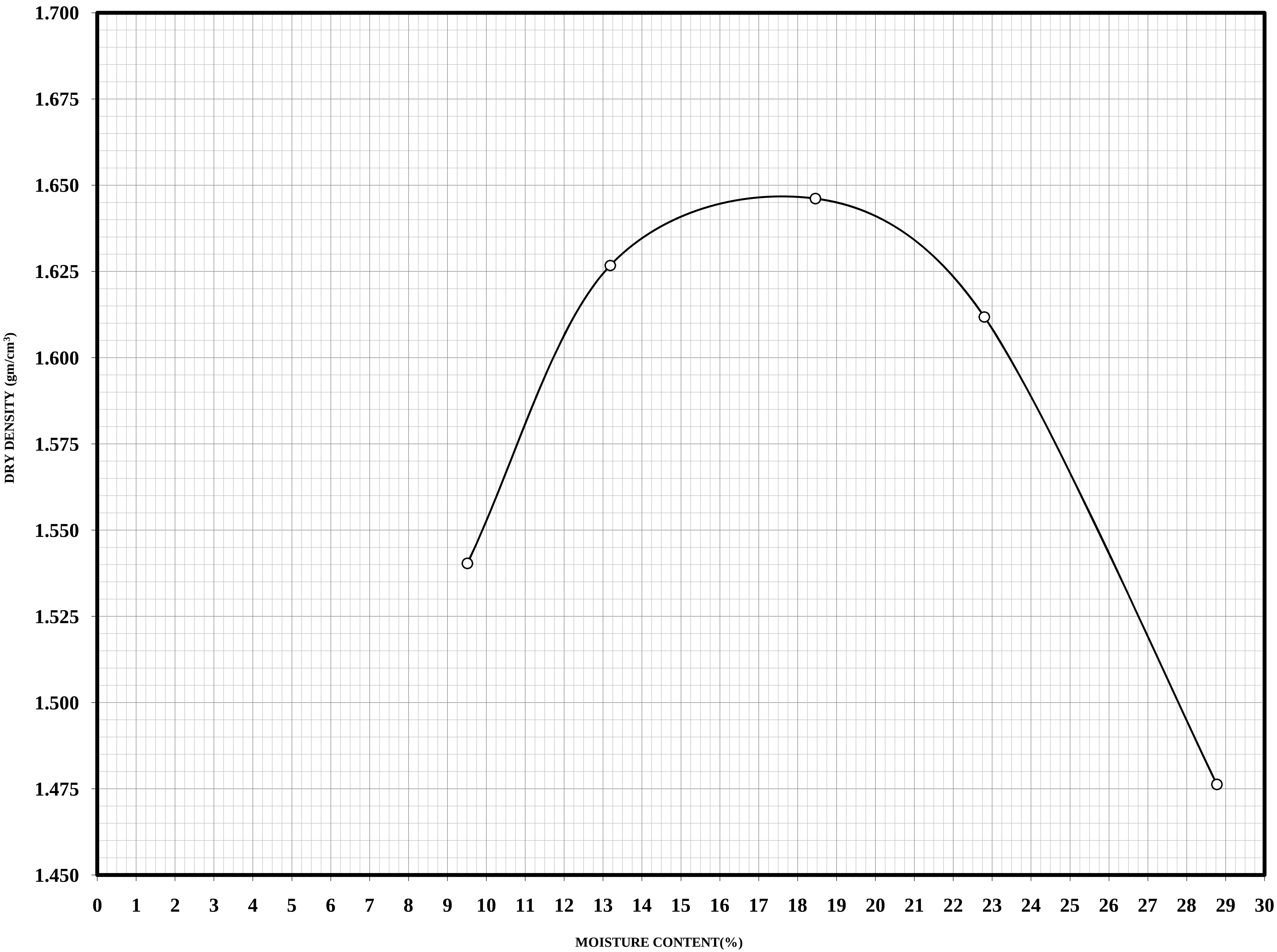
Building & Road Research Institute
University of Khartoum
Cosolidation Test Results
Embankment
B.H #3 - Depth 9m

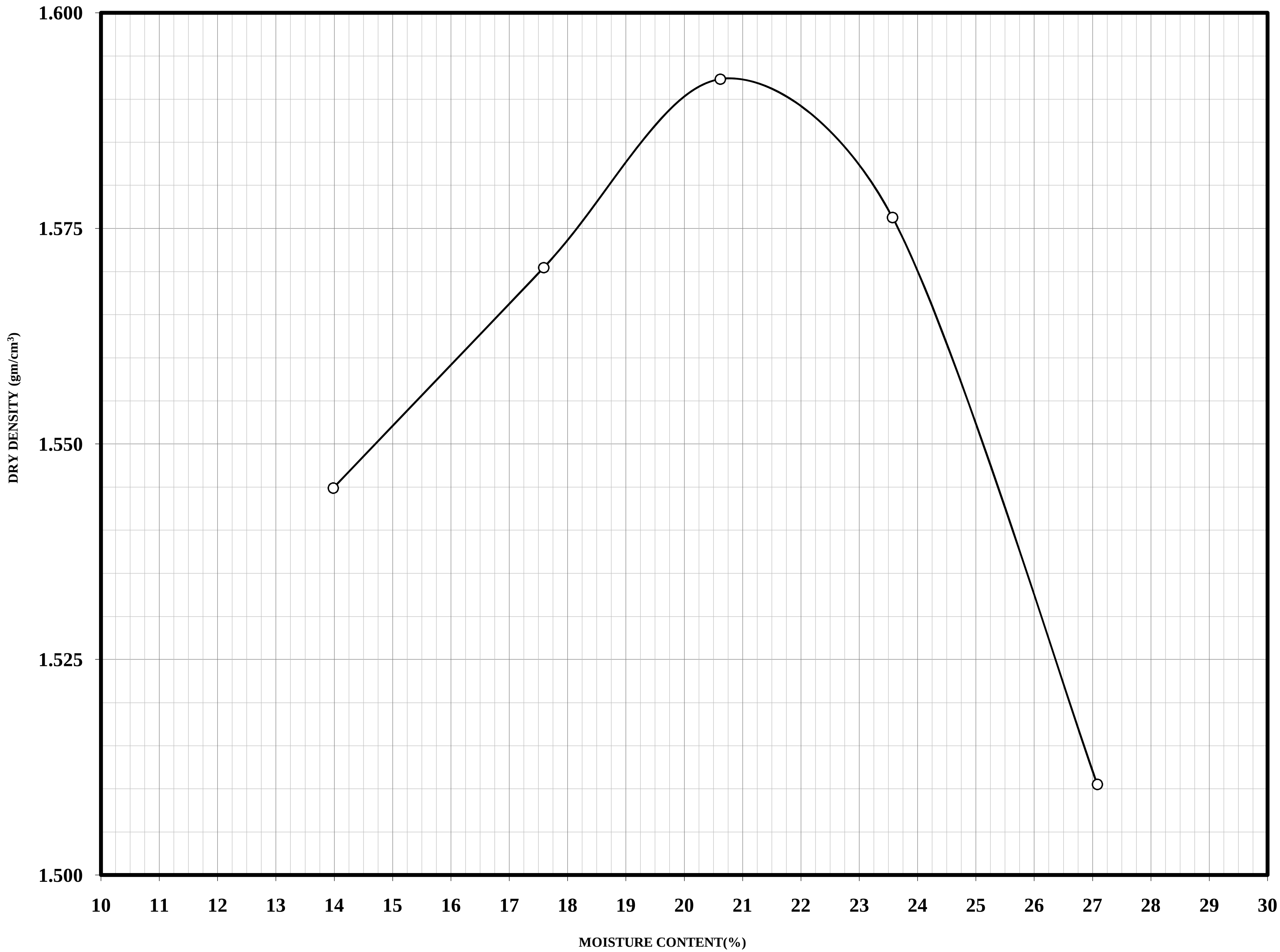


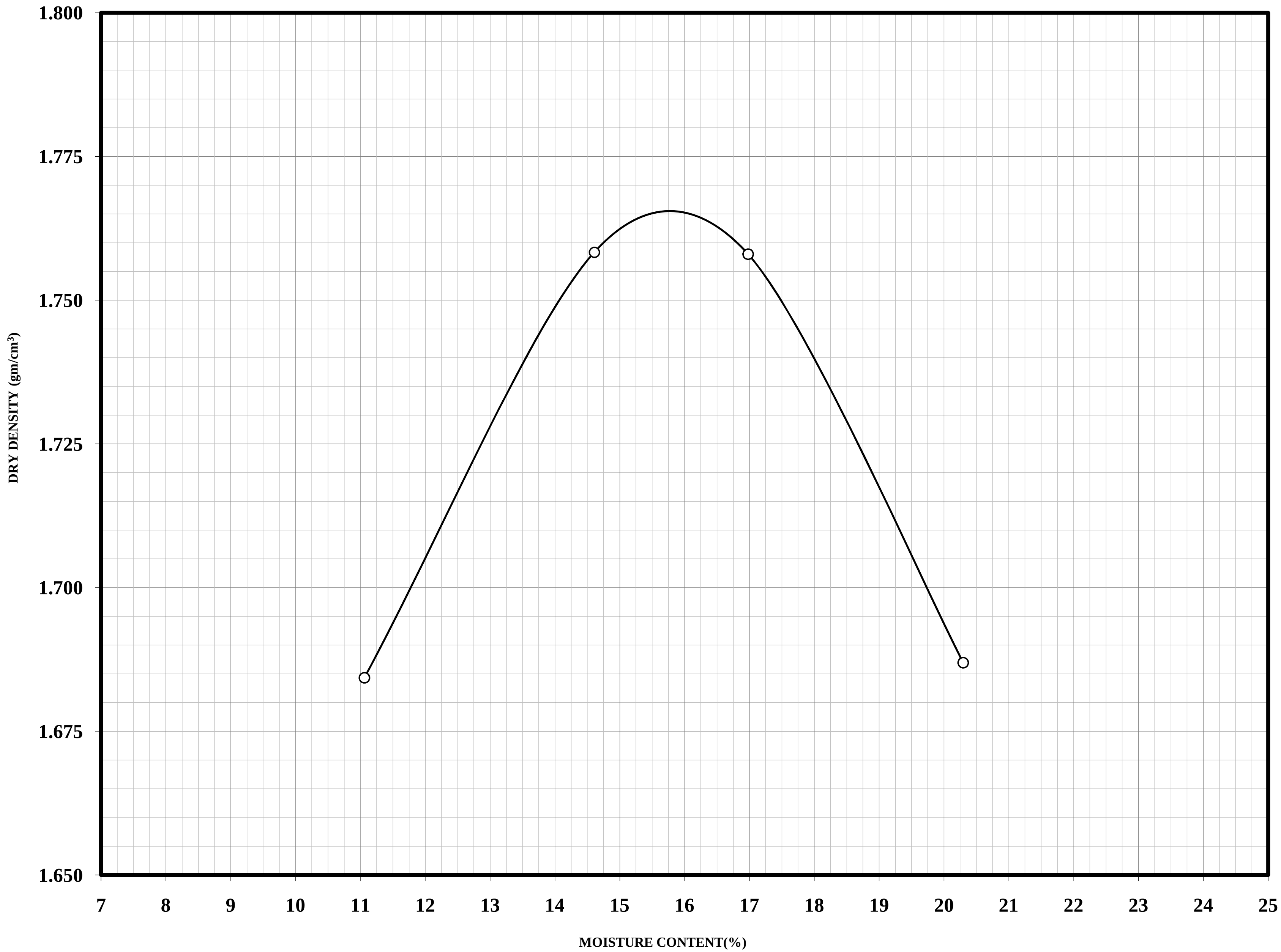

Building & Road Research Institute
University of Khartoum
Cosolidation Test Results
Embankment
B.H #3 - Depth 10.5m
1.15
1.1
1.05
1
0.95
0.9
0.85
Void Ratio
σ Pc = 100 kN/m2.
Cc = 0.1241.
Cr = 0.0219.
0.1
1
10
100
1000
10000
Applied Pressure(kN/m2)

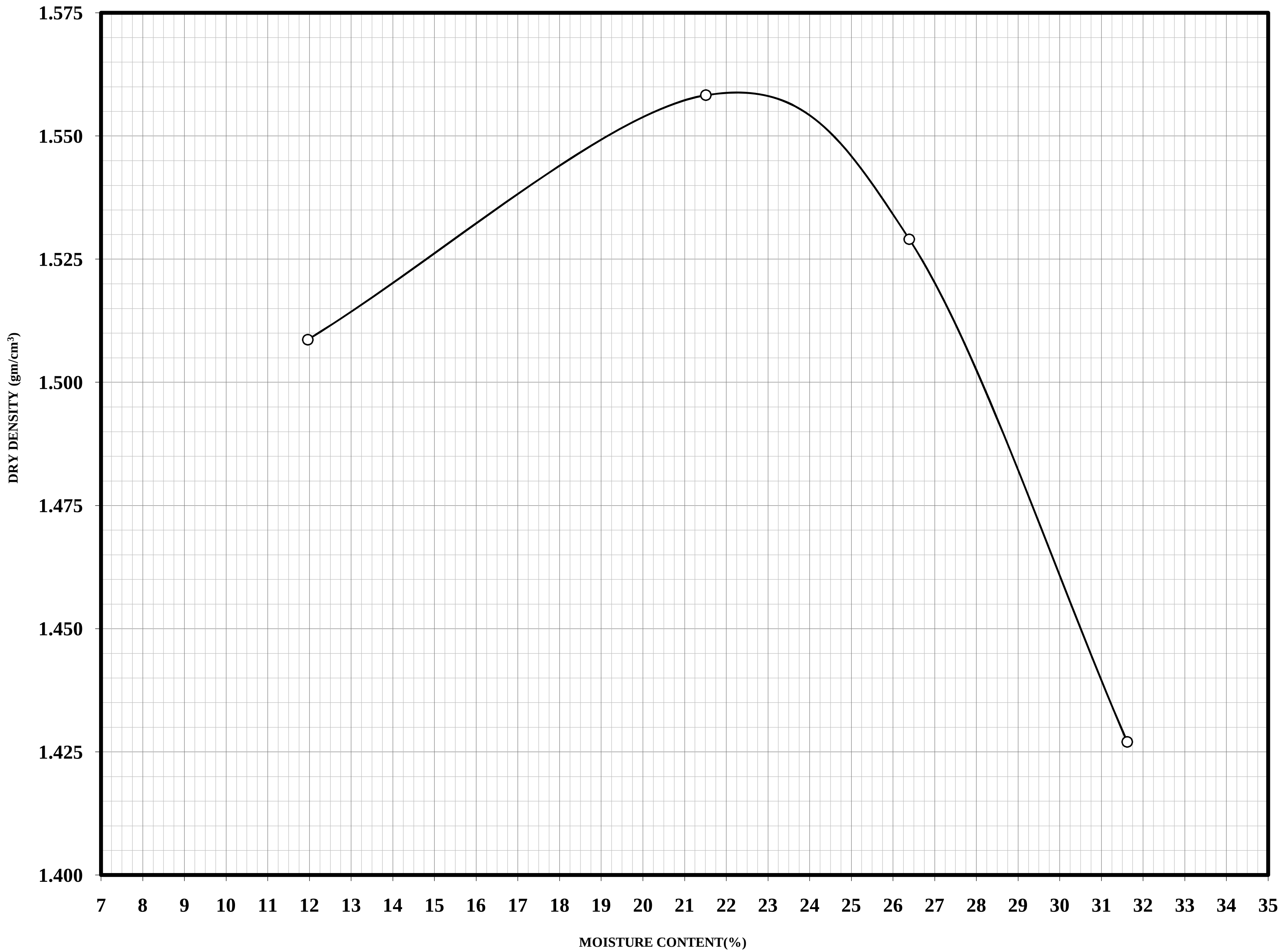
APPENDIX (G)

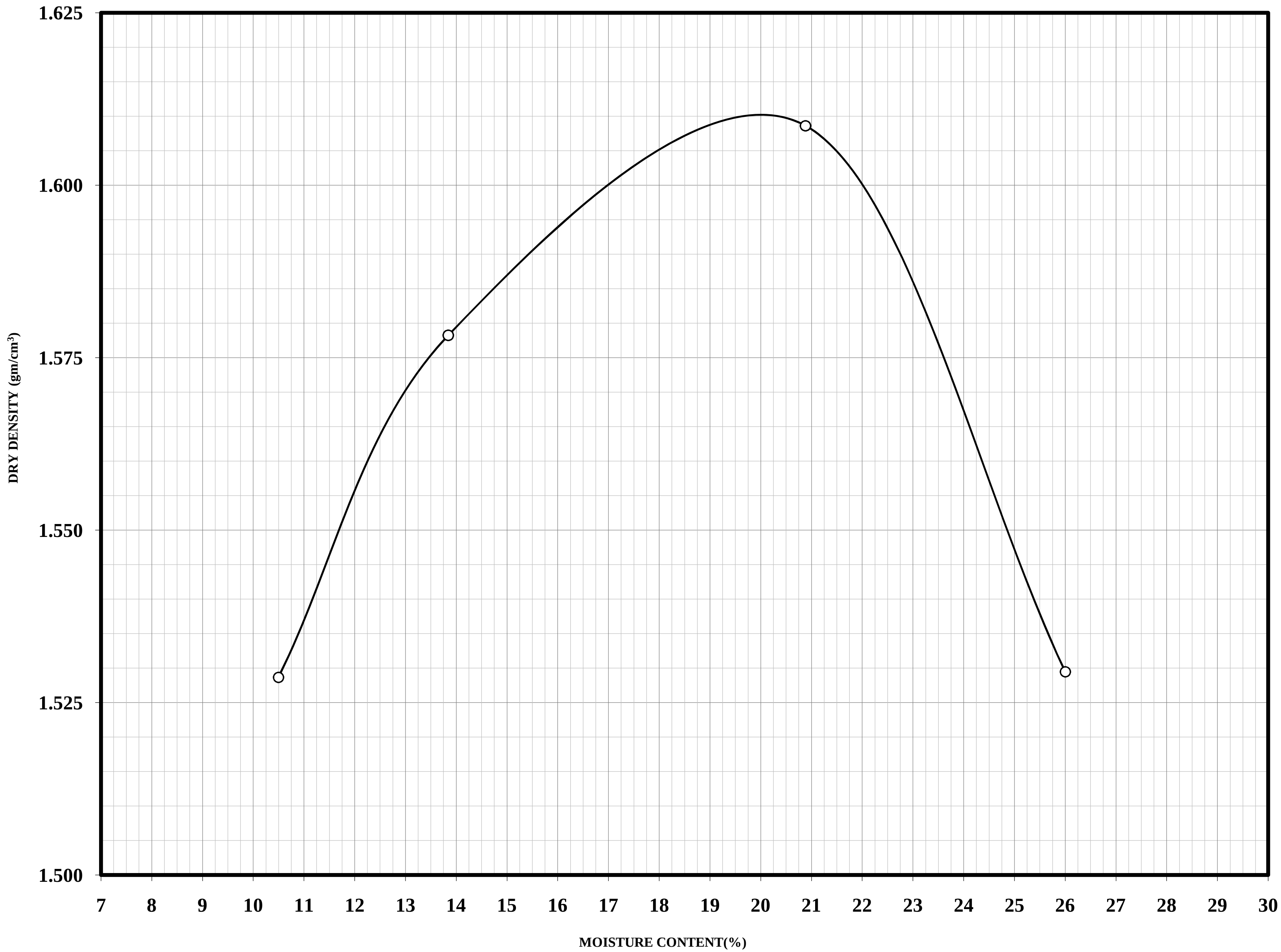
COMPACTION TEST RESULTS

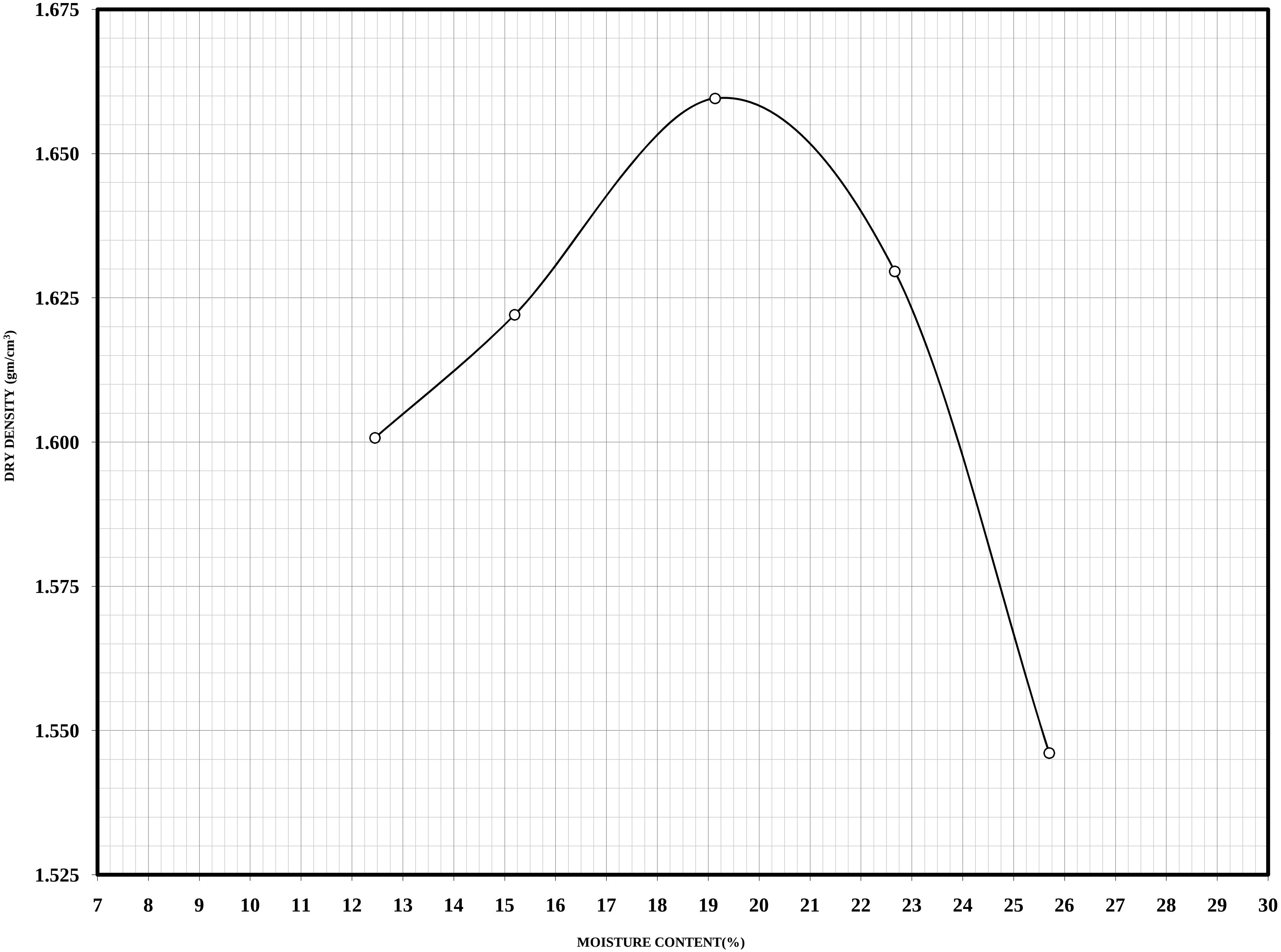

1.700
1.675
1.650
1.625
1.600
1.575
1.550
1.525
1.500
1.475
1.450
DRY DENSITY (gm/cm³)
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
MOISTURE CONTENT(%)


1.600
1.575
1.550
1.525
1.500
DRY DENSITY (gm/cm3)
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
MOISTURE CONTENT(%)


1.800
1.775
1.750
1.725
1.700
1.675
1.650
DRY DENSITY (gm/cm3)
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
MOISTURE CONTENT(%)


1.575
1.550
1.525
1.500
1.475
1.450
1.425
1.400
DRY DENSITY (gm/cm³)
7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35
MOISTURE CONTENT(%)


1.625
1.600
1.575
1.550
1.525
1.500
DRY DENSITY (gm/cm3)
7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
MOISTURE CONTENT(%)



DRY DENSITY (gm/cm3)
1.675
1.650
1.625
1.600
1.575
1.550
1.525
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
MOISTURE CONTENT(%)

APPENDIX (I)

PHOTOGRAPHS TAKEN FROM EXISTING EMBANKMENT

Plate 1: General view of the embankment

Plate 2: General view of the U/S slope of the embankment

Plate 3: General view of the D/S slope of the embankment

Plate 4: General view of the embankment crest

Plate 5: Circular portion of the embankment (U/S view)

Plate 6: Damage in D/S vertical rip-rap wall in the circular portion of the embankment

Plate 7: Difference in level between circular portion crest and drainage hole

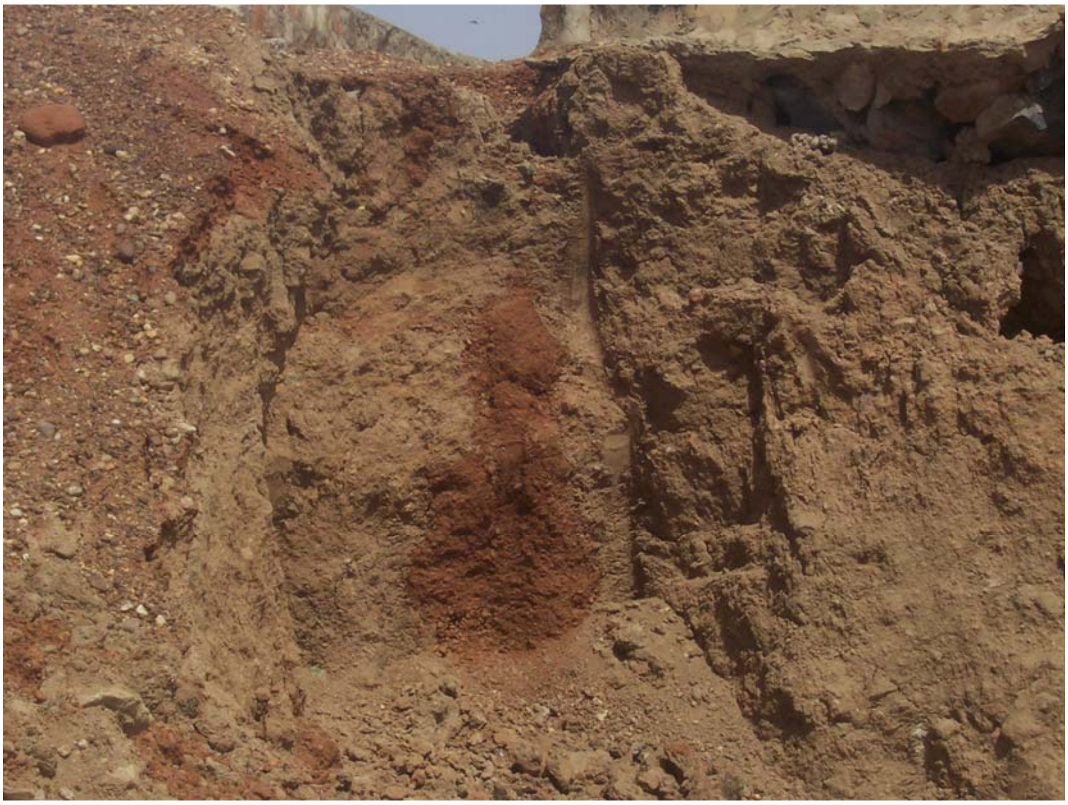


Plate 8: The probability of presence of random soil pockets in the embankment body

Plate 9: Cracks developed in the protection rip-rap of U/S face of the embankment

Plate 10: Cracks developed in the protection rip-rap of U/S face of the embankment

Plate 11: Erosion effects on D/S face of the embankment

Plate 12: Erosion effects on D/S face of the embankment